CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

NOVEMBER 1957 VOL. LII. NO. 11

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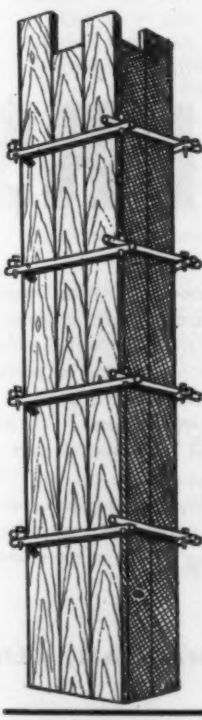

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CLOSING DATE—1st April, 1958.

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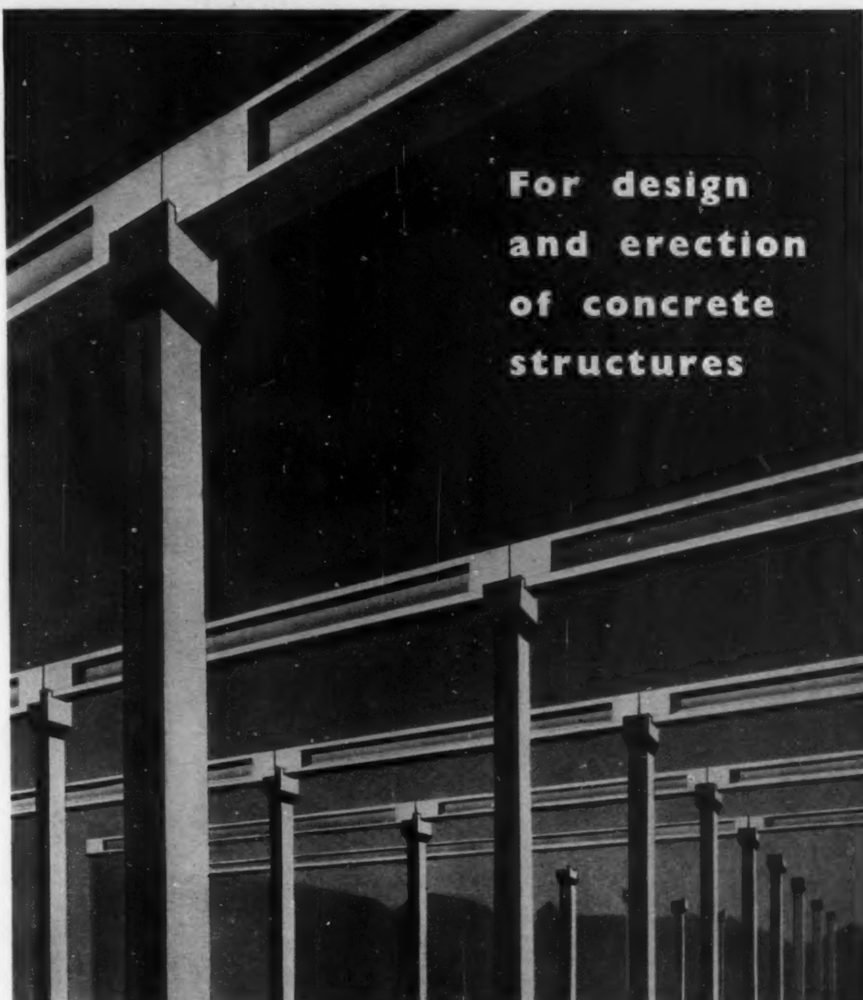
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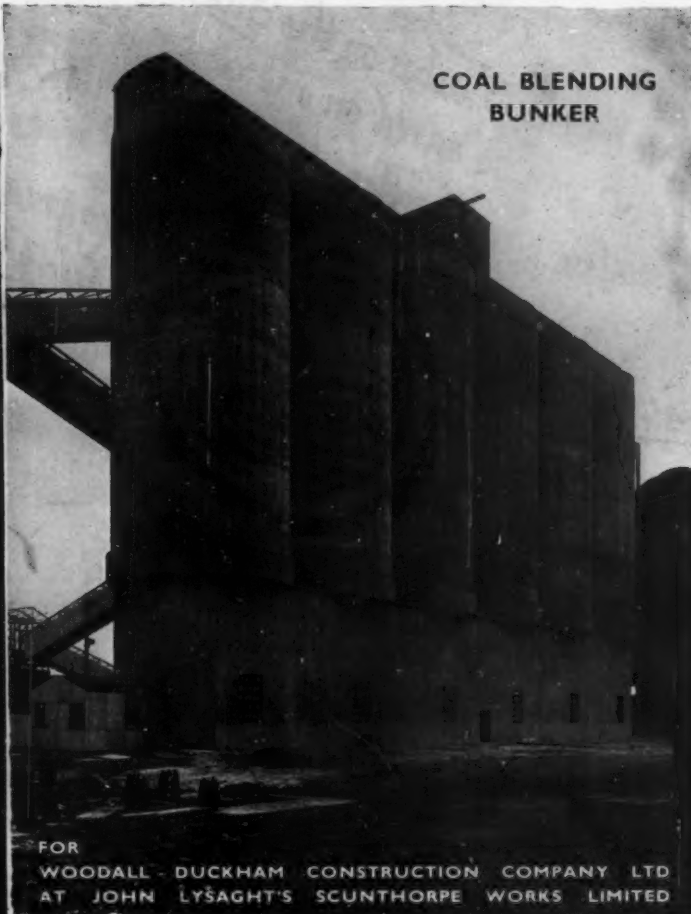
of concrete
structures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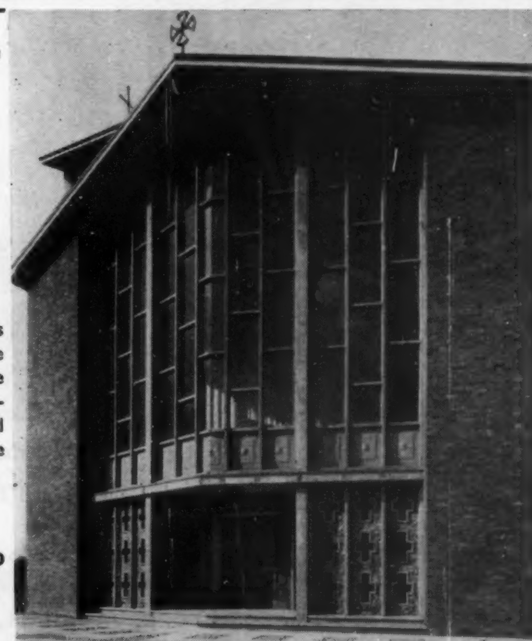

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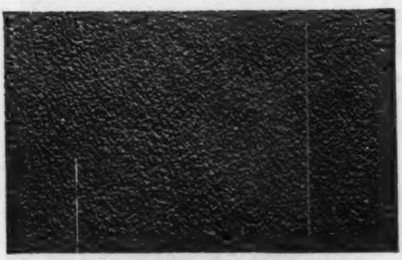
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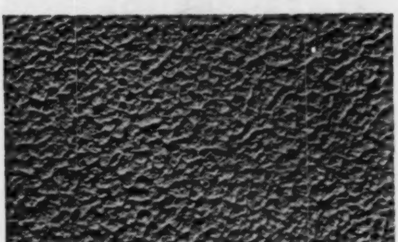
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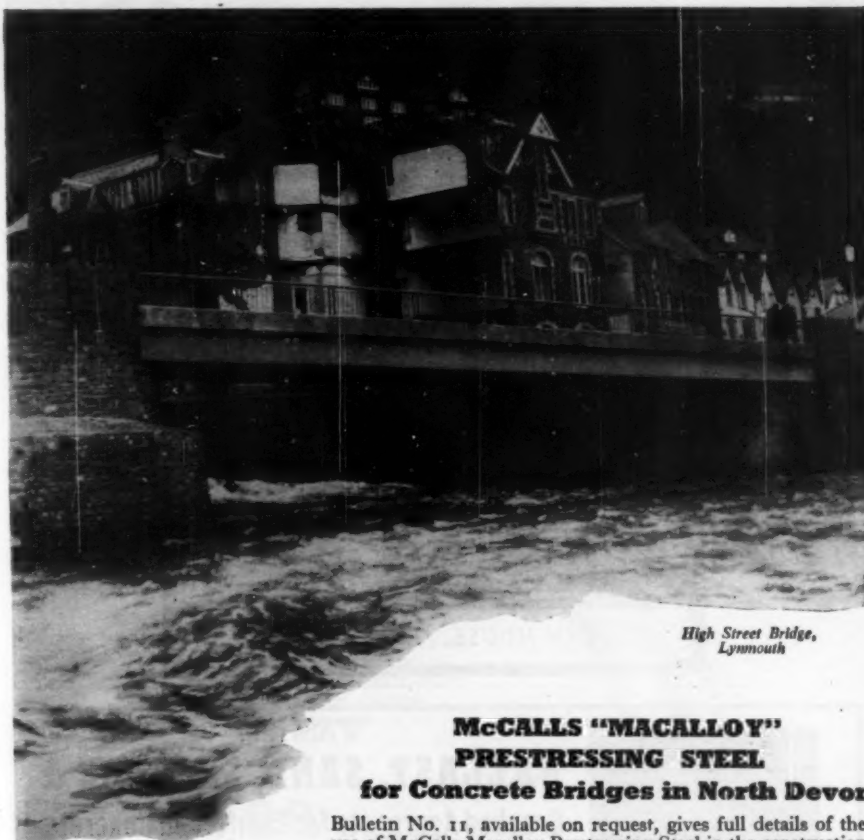

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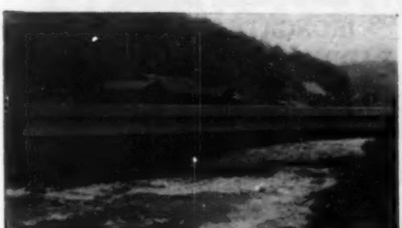

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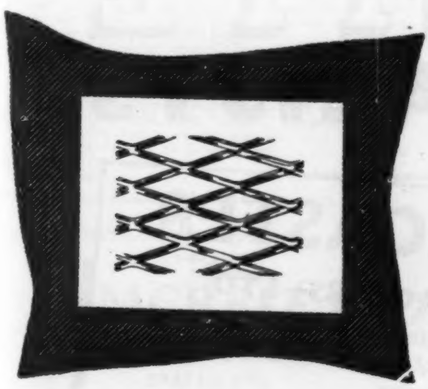

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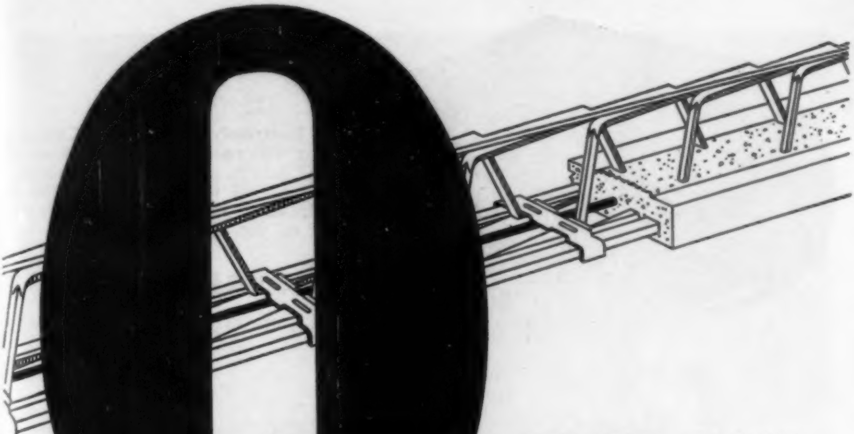


CONCRETE AND CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

Volume LII, No. 11. LONDON, NOVEMBER, 1957.

EDITORIAL NOTES

"Half-educated Engineers."

IN this number (page 391) is published a paper in which an engineer agrees with the comments we have so often made on the lack of education of many young men trained in engineering. " Engineering graduates ", he writes, " are being turned out who are only half educated in any real sense. Some are almost illiterate, fitted no doubt to become excellent back-room boys, but little more than that." When some years ago we wrote " It is common to assume that every man who is awarded a science degree is a scientist or an engineer, although he may never be fitted for any more important duties than those of a laboratory assistant or a designer-draughtsman ", we were strongly contradicted and even abused; for example, a professor of civil engineering wrote, " I pass over the sneers which disfigure your writing . . . it is incredible that the editor of a reputable professional journal . . ." and much more in the same strain. It is good to know that some engineers are now expressing concern on this problem, and perhaps more important to know that the Ministry of Education has stated: " Engineering students have been conditioned to concentrate narrowly on taking as many technological subjects in the shortest possible time in order to obtain the maximum exemption from professional examinations. This is undesirable." The Ministry recommends particularly that students of technical subjects be instructed in the arts of clear thinking and precise expression.

The writer of the paper suggests only a study of history for broadening the mind, but logic and philosophy are perhaps more important in the pursuit of clear reasoning. Such studies would certainly have prevented him from making some illogical statements and unwarranted claims, and even from stating that engineering students are " turned out ". For example, having expressed the view that engineering graduates may be half educated, he complains because the nation does not seem to wish its affairs to be controlled by them. In politics we can vote only for the candidates offered to us and, as we see all too clearly, few of those elected to Parliament have the wisdom that is the most essential requirement of those in control of the affairs of the country. It is not true, as the author states, that we prefer to be governed by politicians, lawyers, and accountants; these people are in Parliament only because a political party nominates them, and the voters have no other choice. In industry, in business,

and wherever there is free choice it is not to be expected that half-educated men will be appointed to high administrative posts. The results are, as the author says, that the engineer may often " come in " only when he is called, and that progress is more likely to be guided by people who are more than half educated.

No one will disagree with the author in his plea that more attention be given to the education of engineers so that they may be better qualified to take a larger share in shaping the destiny of the nation, but we entirely disagree with the claim that " as this is an engineers' world, engineers should have a leading part in directing it ". Engineering should be the servant, not the master, of humanity. Moreover, the engineer is trained to control the forces of nature, which are generally predictable; an entirely different approach is necessary to deal with human beings, whose actions and reactions are never predictable and cannot be ascertained by the use of formulæ or a slide-rule, but whose minds can be influenced by clap-trap as well as by sound argument. We see no reason why engineers generally should make better, or worse, administrators than men in other professions or trades, but a knowledge of engineering does not of itself make a statesman or an administrator.

It is a pity that the author confuses learning with money and " status ". Learning should be pursued for its own sake. It is likely that the greater a man's learning the happier he will be, the more valuable he will be as a citizen and in his profession, the greater will be his reward in the esteem of his fellows, and the more likely he is to be selected for an administrative post. It is true that no employer is likely to increase the salary of an engineer merely because he has a diploma in history (and some diplomas are gained so easily as to be meaningless), but if his studies have made him a fuller and wiser man then there is every possibility that he will attain a higher position. It is quite a different matter to threaten that, if all engineers had a diploma in history, " What individuals cannot accomplish for themselves a whole profession can ", and that the possession of these diplomas would give engineers the right to demand a leading part in directing the affairs of the world. The same claim could be made for any other profession or trade that could persuade most of its members to obtain diplomas in history.

A study of history and of the influence of engineering on the development of society is well worth while, but such studies cannot of themselves make good governors or good administrators. For these positions the essential requirement is wisdom—and wisdom does not depend upon a knowledge of engineering or of history. A study of the arts, of the humanities, of history, and of science, and, most important of all, natural ability, will help in the pursuit of wisdom; they are a means to an end, and if the end can be better achieved by engineers than by people in other walks of life, then indeed engineers will be in a sounder position to aspire to rule the nation. But it may well be that, with greater wisdom, engineers may prefer to keep out of politics and administration and to seek their pleasure in the work to which they have dedicated their lives, that is the control of the forces of nature rather than office management, sales promotion, and controlling the destiny of the nation. The salaries paid to engineers are now comparable with those paid in other professions, and there is no doubt that rewards will keep pace with the prosperity of the nation and with the demand for the services of engineers. Engineers who have acquired more wisdom than their fellows will, in future as in the past, be selected for more remunerative posts. It is not ordained that all men should have equal talents.

Design of Eccentrically-Loaded Columns by the Load-Factor Method.

1.—Symmetrical Cold-worked Reinforcement.

By J. D. BENNETT, B.Eng.

THE British Standard Code of Practice No. 114 (1957) permits the design of eccentrically-loaded columns by ultimate-load methods and gives formulæ which may be used. These are rather involved, and the formula for failure in compression is conservative in that the percentage of cold-worked steel required for certain loads and eccentricities is greater than if mild steel is used at a lower stress. In this article formulæ are developed and graphs are given for columns with symmetrical cold-worked reinforcement so that the process of design is short and simple. It is proposed to publish charts for columns reinforced with mild steel, and for columns with unsymmetrical reinforcement, in future numbers of this journal.

NOTATION.—With the exceptions of n and p_t all the symbols listed are the same as those used in the Code of Practice.

A_s : total cross-sectional area of steel.
A_{sc} : cross-sectional area of steel in compression.
A_{st} : cross-sectional area of steel in tension.
b : breadth of column.
d : overall depth of column.
d_1 : effective depth to the tensile reinforcement.
d_2 : depth to the compressive reinforcement.
e : eccentricity of a load on a column.
E_s : secant modulus of elasticity for cold-worked bars.
M : bending moment ($= Pe$).
nd_1 : depth to neutral axis.
P : permissible load on a short column subjected to both direct load and bending.
P_b : load on a column as defined by equation (1).
p_{cc} : permissible stress in concrete in direct compression.
p_{sc} : permissible compressive stress in the reinforcement.
p_{st} : permissible tensile stress in the reinforcement.
p_t : stress in tensile reinforcement at ultimate load.
r : ratio of steel $= \dfrac{A_s}{bd}$.
u : cube crushing strength of the concrete.
Xd_1 : depth of assumed stress-block.

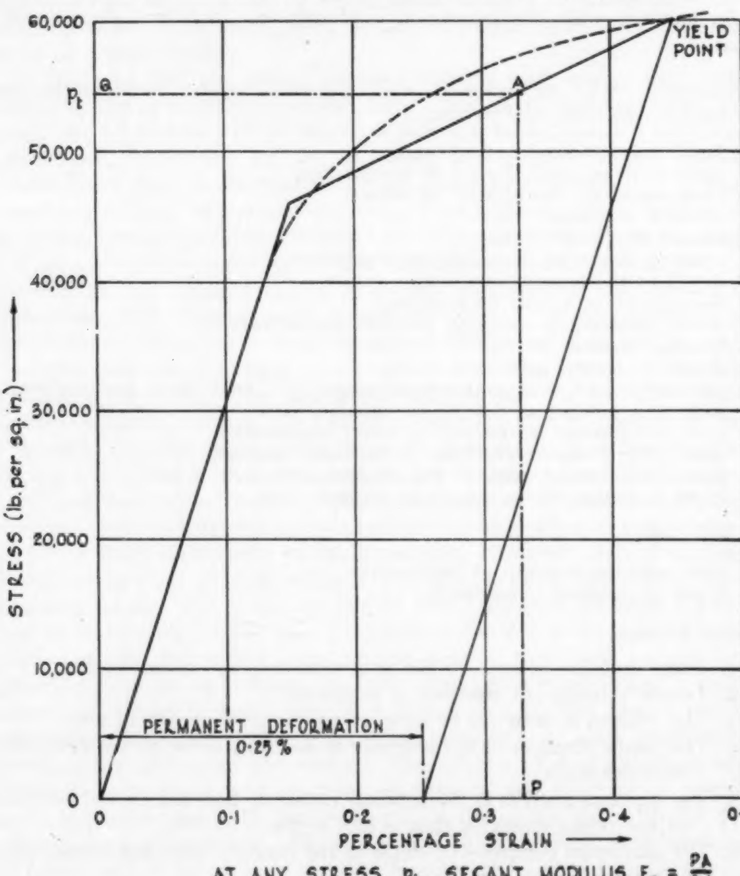
BASIS OF DESIGN.

1. Plane sections normal to the axis remain plane after bending.
2. Tensile strength of concrete is neglected.
3. The column is designed to have a load-factor generally of 2·0.
4. The prism strength of the concrete is assumed to be 76 per cent. of the cube strength.
5. The maximum stress in the concrete at failure does not exceed two-thirds of the prism strength, that is $\frac{2}{3} \times 0{\cdot}76u$.
6. The maximum compressive strain in the concrete does not exceed $\frac{1}{300}$ at failure.

7. The distribution of strain across the section is linear.
8. The distribution of compressive stress in the concrete at failure is rectangular.
9. The depth of the rectangular stress-block is 0·85 × depth of neutral axis.
10. The secant modulus of elasticity for cold-worked steel at any stress p_t is given by $\frac{PA}{QA}$ in *Graph* 1.

Assumptions 1 to 9 are either stated or implied in the Code, which also requires the use of the secant modulus of elasticity. In the following analysis for failure due to compression the actual stress in the tensile steel at ultimate load is calculated and the corresponding secant modulus from *Graph* 1 is used. The stress-strain

Graph No. I. Assumed Stress-Strain Curve for Cold Worked Bars.



curves for cold-worked bars with a minimum yield-stress of 60,000 lb. per square inch vary considerably, and only an occasional sample would have a yield-point as low as 60,000 lb. per square inch. A bar with a yield-point of exactly 60,000 lb. per square inch would have a stress-strain curve indicated by the broken line on *Graph* 1. Thus the two straight lines assumed for the purpose of obtaining the secant modulus will give safe values. The limit of proportionality is assumed to be 46,000 lb. per square inch, and the minimum yield at a strain of 0·45 per cent. corresponds to the permanent deformation of 0·25 per cent. specified in clause 304 of the Code.

If P_b is the balancing load between failure in tension and failure in compression, then failure will occur when the maximum strain in the concrete is reached simultaneously with the maximum stress in the tensile steel. At this balanced condition the following conditions will exist:

Strain in concrete $= \dfrac{1}{300}$. Stress in steel $= 2p_{st}$. Secant modulus of elasticity $= E_s$.

From the linear strain relationship, $\dfrac{2p_{st}}{E_s(1-n)} = \dfrac{1}{300n}$.

$$\text{Therefore } n = \frac{100{,}000}{100{,}000 + \dfrac{60p_{st} \times 10^6}{E_s}}.$$

The depth of the assumed stress block is $0{\cdot}85nd_1 = Xd_1$, and

$$X = \frac{85{,}000}{100{,}000 + \dfrac{60p_{st} \times 10^6}{E_s}}.$$

The permissible stress in the concrete is $\frac{1}{2} \times 0{\cdot}76 \times \frac{2}{3}\, u = p_{cc}$.

$$P_b = (p_{cc}bd_1X) - A_{st}p_{sc} + A_{sc}p_{sc} \qquad (1)$$

If the load exceeds P_b the section will fail in compression, and if the load is less than P_b the section will fail in tension.

Failure in compression.—P greater than P_b. Column with symmetrical reinforcement $r = \dfrac{A_s}{bd} = \dfrac{2A_{st}}{bd} = \dfrac{2A_{sc}}{bd}$. The permissible stresses and the strains at ultimate load are as shown in *Fig.* 1. The permissible stresses assume a load-factor of two, and are used in the following equations of equilibrium.

Resolving forces axially, $P = p_{cc}bd_1X + A_{sc}p_{sc} - A_{st}\left(\dfrac{1-n}{n}\right)\dfrac{E_s}{600}$, giving

$$\frac{P}{bdp_{cc}} = 0{\cdot}85n\frac{d_1}{d} + \frac{r}{p_{cc}}.\frac{p_{sc}}{2} - \frac{r}{p_{cc}}\left(\frac{1-n}{n}\right)\frac{E_s}{1{,}200} \qquad (2)$$

Calculating moments about the centroid of the tensile steel,

$P\left(e + \dfrac{d_1 - d_2}{2}\right) = p_{cc}bd_1{}^2X\left(1 - \dfrac{X}{2}\right) + A_{sc}p_{sc}\,(d_1 - d_2)$, from which

$$\frac{M}{bd^2p_{cc}} + \frac{P}{bdp_{cc}}.\frac{d_1 - d_2}{2d} = 0{\cdot}85\left(\frac{d_1}{d}\right)^2 n(1 - 0{\cdot}425n) + \frac{r}{p_{cc}}.\frac{p_{sc}}{2}.\frac{d_1 - d_2}{d}. \qquad (3)$$

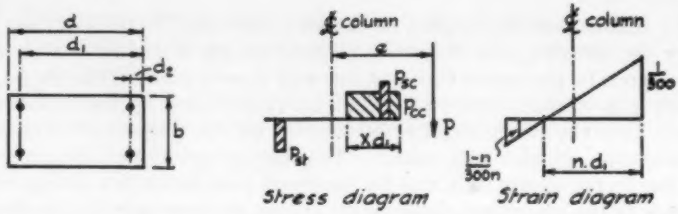


Fig. 1.

The strain in the tensile steel at ultimate load is $\frac{1-n}{300n}$, and E_s is determined from *Graph* 1 for various values of n.

For the range $p_t = 60{,}000$ to $46{,}000$ lb. per square inch, $n = 0{\cdot}426$ to $0{\cdot}685$, and $E_s = 13\frac{1}{3} \times 10^6$ to 30×10^6 lb. per square inch.

For the range $p_t = 46{,}000$ to 0 to $-46{,}000$ lb. per square inch, $n = 0{\cdot}685$ to 1 to $1{\cdot}85$ and $E_s = 30 \times 10^6$ lb. per square inch.

If n is less than $0{\cdot}426$ the section will fail in tension.

Using a fixed cover-ratio and constant values of $\frac{r}{p_{cc}}$, graphs are plotted of $\frac{P}{bdp_{cc}}$ against $\frac{M}{bd^2p_{cc}}$ from equations (2) and (3) for $p_{sc} = 23{,}000$ lb. per square inch.

The foregoing equations do not allow for the reduction in the area of the concrete due to the compressive steel, but this error is small and for a column with 6 per cent. of steel and 1 : 2 : 4 concrete the maximum error is 2·2 per cent. when the column is concentrically loaded. For columns with eccentricities or smaller percentages of steel this error is reduced.

Failure in tension.—P less than P_b. At ultimate load the tensile steel yields at a stress of $2p_{st}$ and it is not necessary to use the linear strain relationship or the secant modulus of elasticity. Resolving forces axially,

$$P = p_{cc}.bd_1X + \frac{r}{2}bd(p_{sc} - p_{st}), \text{ that is}$$

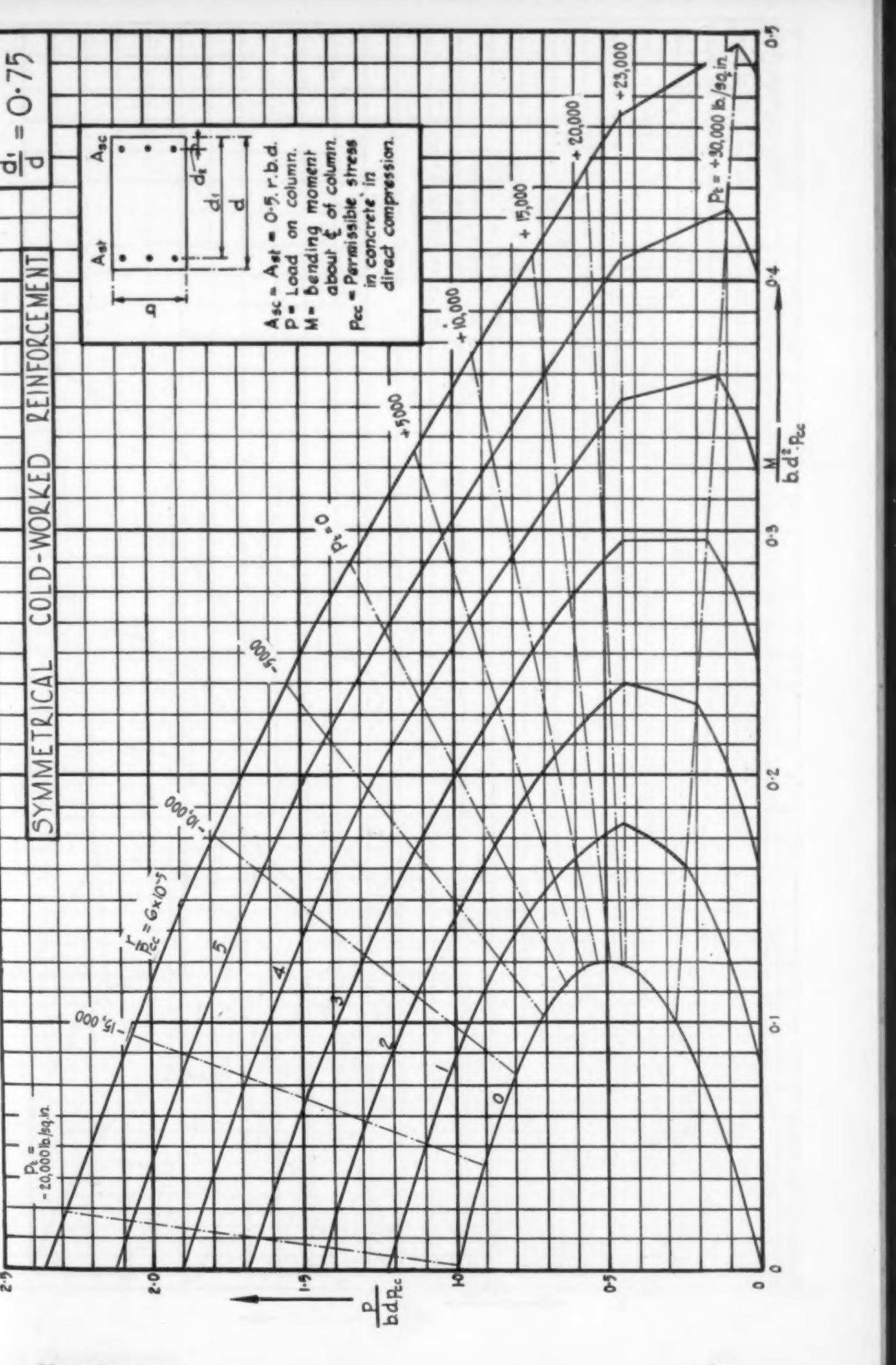
$$\frac{P}{bdp_{cc}} = \frac{d_1}{d}X + \frac{r}{p_{cc}}\left(\frac{p_{sc} - p_{st}}{2}\right) \quad . \quad . \quad . \quad (4)$$

Calculating the moments about the tensile steel,

$$P\left(e + \frac{d_1 - d_2}{2}\right) = p_{cc}bd_1^2\,X\left(1 - \frac{X}{2}\right) + \frac{r}{2}bdp_{sc}(d_1 - d_2), \text{ from which}$$

$$\frac{M}{bd^2p_{cc}} + \frac{P}{bdp_{cc}}\left(\frac{d_1 - d_2}{2d}\right) = \left(\frac{d_1}{d}\right)^2X\left(1 + \frac{X}{2}\right) + \frac{r}{p_{cc}}p_{sc}\left(\frac{d_1 - d_2}{2d}\right) \quad . \quad (5)$$

If X is eliminated between equations (4) and (5) then the formula for tensile failure given in the Code is obtained, but for the purpose of plotting graphs it is more convenient to keep the equations separate. If $p_{st} = 30{,}000$ lb. per square inch and $p_{sc} = 23{,}000$ lb. per square inch, equations (4) and (5) become


SYMMETRICAL COLD-WORKED REINFORCEMENT
$\frac{d_1}{d} = 0{\cdot}75$
$A_{sc} = A_{st} = 0{\cdot}5.r.b.d.$
P = Load on column.
M = Bending moment about ℄ of column.
p_{cc} = Permissible stress in concrete in direct compression.
A_{sc}
A_{st}
d_2
d_1
d
b
$\frac{r}{p_{cc}} = 6 \times 10^{-5}$
$p_t = -20{,}000$ lb./sq.in.
$p_t = +30{,}000$ lb./sq.in.
$p_t = 0$
−15,000
−10,000
−5000
+5000
+10,000
+15,000
+20,000
+23,000
0
1
2
3
4
5
$\frac{P}{b.d.p_{cc}}$
$\frac{M}{b.d^2.p_{cc}}$
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0·4
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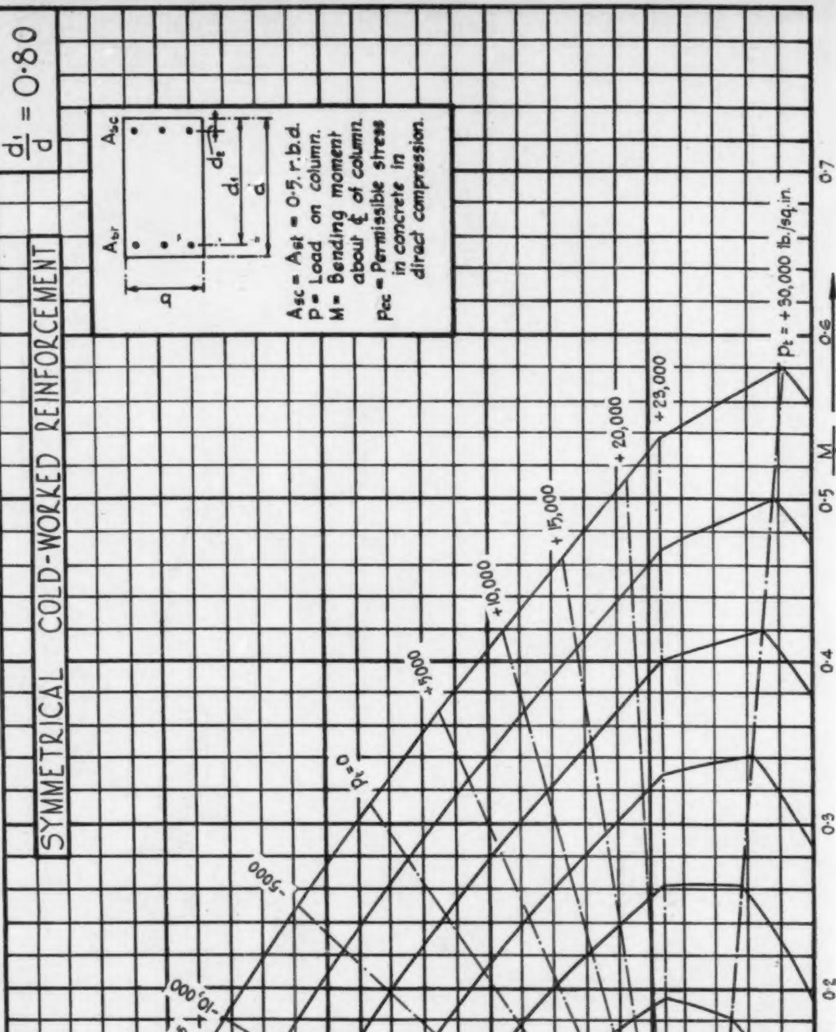
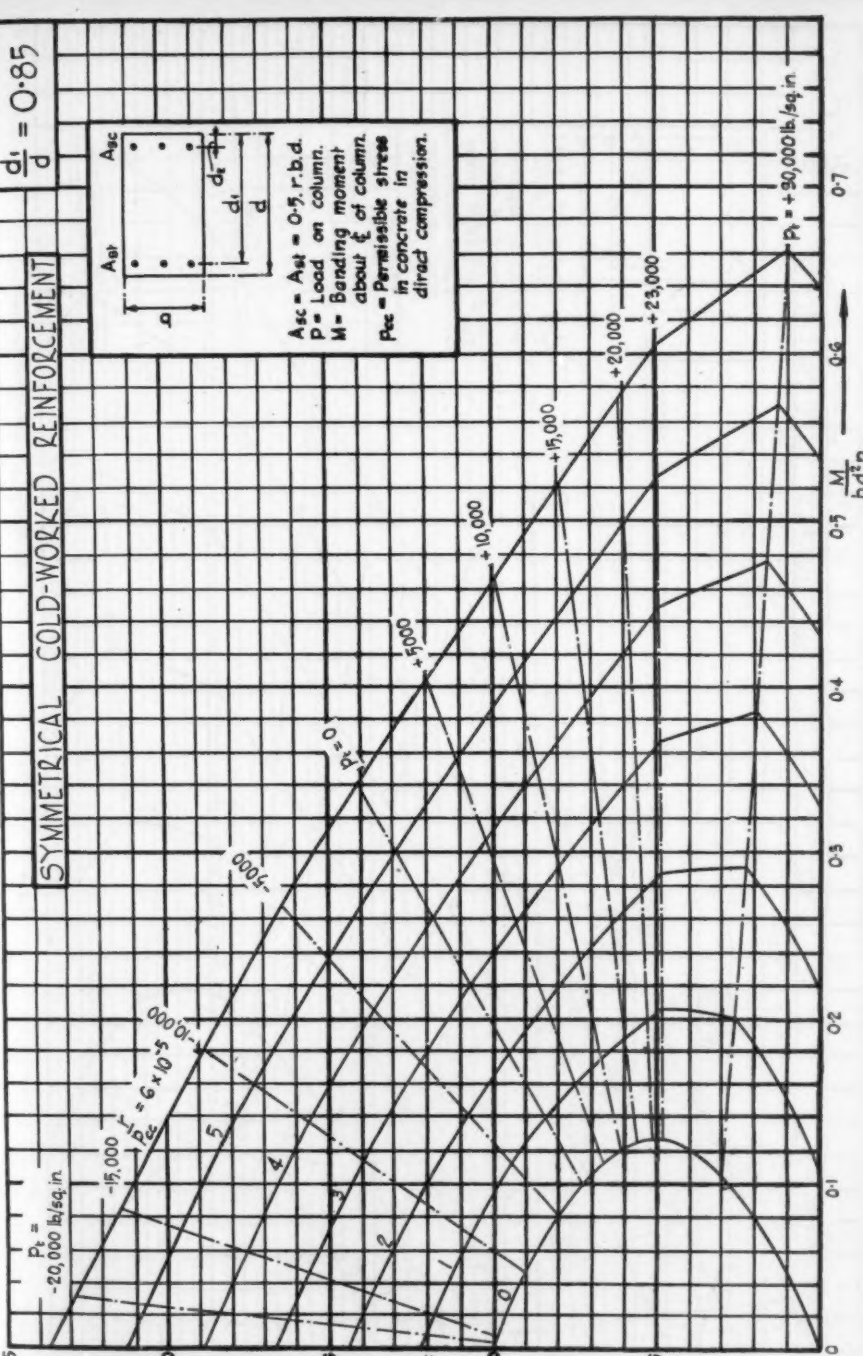
0·5
1·0
1·5
2·0
2·5



SYMMETRICAL COLD-WORKED REINFORCEMENT
$\frac{d_1}{d} = 0{\cdot}80$
$A_{sc} = A_{st} = 0{\cdot}5\ r.b.d.$
P = Load on column.
M = Bending moment about ℄ of column.
p_{cc} = Permissible stress in concrete in direct compression.
p_t = −20,000 lb/sq.in.
p_t = +30,000 lb./sq.in.
$\frac{r}{p_{cc}} = 6 \times 10^{-3}$
−15,000
−10,000
−5000
$p_t = 0$
+5000
+10,000
+15,000
+20,000
+23,000
$\frac{P}{b.d.p_{cc}}$
$\frac{M}{b.d^2 p_{cc}}$


SYMMETRICAL COLD-WORKED REINFORCEMENT
$\frac{d_1}{d} = 0{\cdot}85$
$A_{sc} = A_{st} = 0{\cdot}5.r.b.d.$
P = Load on column.
M = Bending moment about ℄ of column.
p_{cc} = Permissible stress in concrete in direct compression.
A_{st}
A_{sc}
b
d_1
d_2
d
$p_t = -20{,}000$ lb/sq.in.
$-15{,}000$
$-10{,}000$
-5000
$p_t = 0$
$+5000$
$+10{,}000$
$+15{,}000$
$+20{,}000$
$+23{,}000$
$p_t = +30{,}000$ lb./sq.in.
$\frac{r}{p_{cc}} = 6 \times 10^{-5}$
0
1
2
3
4
5
$\frac{P}{b.d.p_{cc}}$
$\frac{M}{b.d^2.p_{cc}}$
0
0·5
1·0
1·5
2·0
2·5
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0·4
0·5
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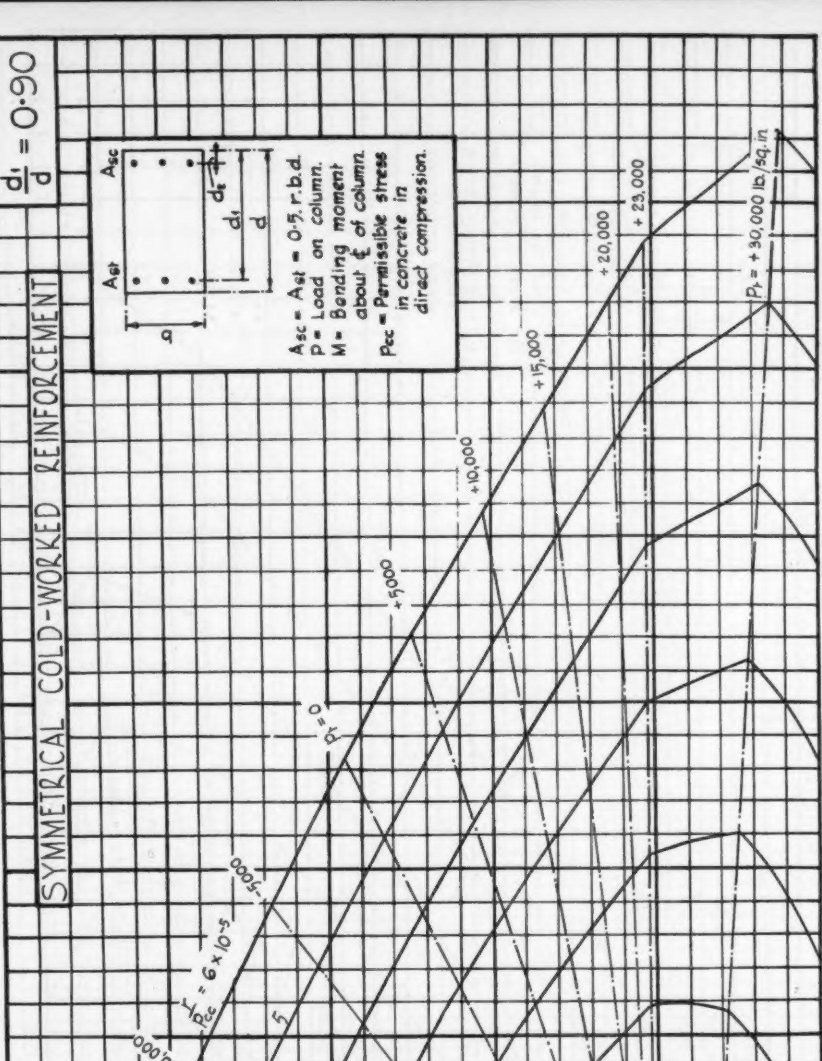
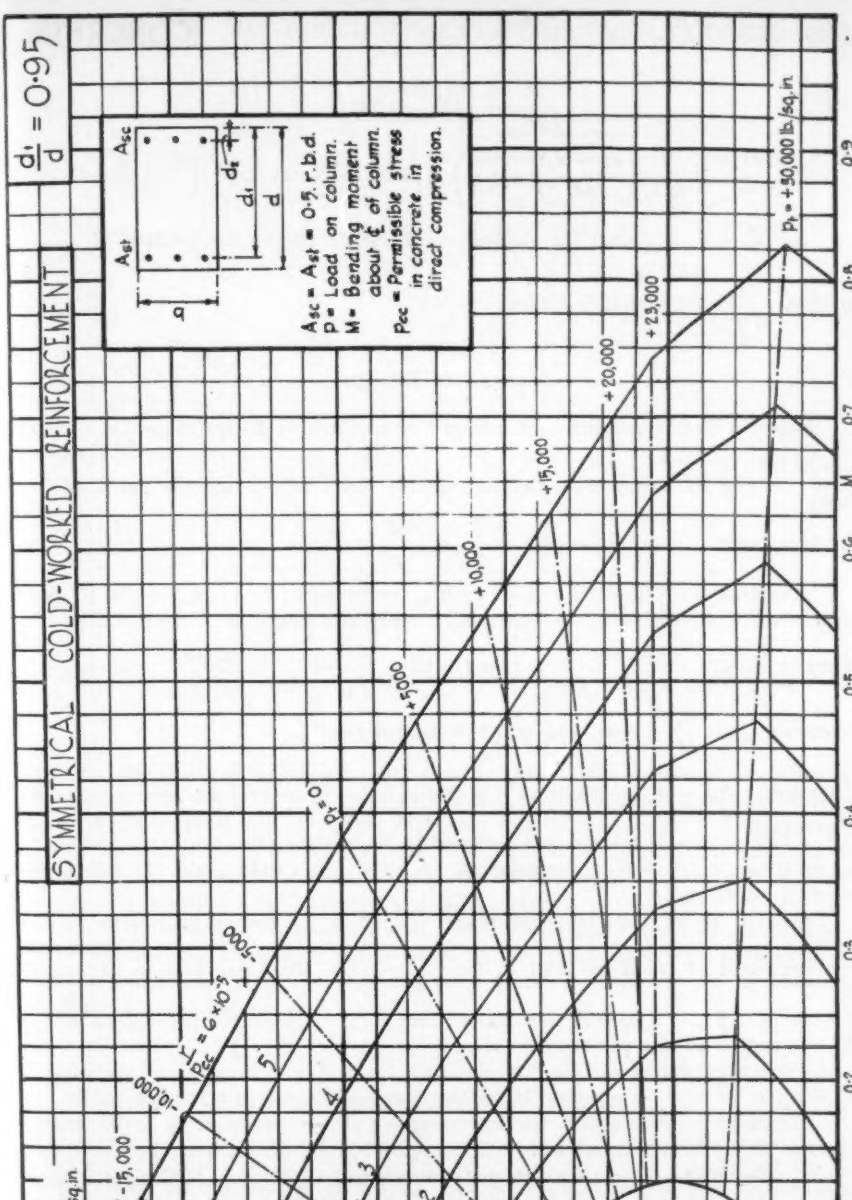

SYMMETRICAL COLD-WORKED REINFORCEMENT
$\frac{d_1}{d} = 0{\cdot}90$
Asc = Ast = 0·5. r. b. d.
P = Load on column.
M = Bending moment about ℄ of column.
Pcc = Permissible stress in concrete in direct compression.
Ast
Asc
b
d1
d
d2
$\frac{r}{p_{cc}} = 6 \times 10^{-5}$
p_t = -20,000 lb./sq. in.
-15,000
-10,000
-5000
$p_t = 0$
+5000
+10,000
+15,000
+20,000
+23,000
p_t = +30,000 lb./sq. in.
0
1
2
3
4
5
$\frac{P}{b.d.p_{cc}}$
$\frac{M}{b.d^2.p_{cc}}$
0
0·1
0·2
0·3
0·4
0·5
0·6
0·7
0·8
0·5
1·0
1·5
2·0
2·5


SYMMETRICAL COLD-WORKED REINFORCEMENT
d1/d = 0·95
Asc = Ast = 0·5. r. b.d.
P = Load on column.
M = Bending moment about ℄ of column.
Pcc = Permissible stress in concrete in direct compression.
P / b.d.Pcc
M / b.d².Pcc
pt = -20,000 lb./sq.in.
-15,000
-10,000
-5000
pt = 0
+5,000
+10,000
+15,000
+20,000
+23,000
pt = +50,000 lb./sq.in.
r / Pcc = 6×10⁻⁵
5
4
3
2
1
0

$$\frac{P}{bdp_{cc}} = \frac{d_1}{d}X - 3500\frac{r}{p_{cc}} \quad . \quad . \quad . \quad . \quad (6)$$

$$\frac{M}{bd^2p_{cc}} + \frac{P}{bdp_{cc}}\left(\frac{d_1 - d_2}{2d}\right) = \left(\frac{d_1}{d}\right)^2 X\left(1 - \frac{X}{2}\right) + 11{,}500\frac{r}{p_{cc}}\left(\frac{d_1 - d_2}{d}\right) \quad . \quad (7)$$

For $d_1 + d_2 = d$ and for various cover-ratios curves are plotted of $\frac{P}{bdp_{cc}}$ against $\frac{M}{bd^2p_{cc}}$ for constant values of $\frac{r}{p_{cc}}$.

Method of Design.

Select a size of column and mixture of concrete and determine $\frac{M}{bd^2p_{cc}}$ and $\frac{P}{bdp_{cc}}$. Using the graph with the appropriate cover ratio, read the value of $\frac{r}{p_{cc}}$ for this point. Then $A_{st} = A_{sc} = \frac{r}{p_{cc}}.p_{cc}.\frac{bd}{2}$.

EXAMPLE 1.—Column 24 in. × 12 in.; concrete 1 : 2 : 4; $p_{cc} = 760$ lb. per square inch; direct load $P = 130{,}000$ lb.; bending moment $M = 1{,}300{,}000$ in.-lb.

Then $\frac{P}{bdp_{cc}} = \frac{130{,}000}{12 \times 24 \times 760} = 0{\cdot}595$, and $\frac{M}{bd^2p_{cc}} = \frac{1{,}300{,}000}{12 \times 24^2 \times 760} = 0{\cdot}247$.

Assuming that $\frac{d_1}{d} = 0{\cdot}90$, the graph indicates that $\frac{r}{p_{cc}} = 1{\cdot}5 \times 10^{-5}$.

Therefore $A_{sc} = A_{st} = 1{\cdot}5 \times 10^{-5} \times 760 \times \frac{12 \times 24}{2} = 1{\cdot}64$ sq. in.

EXAMPLE 2.—If the column designed in Example 1 with 1·64 sq. in. of steel on each face is checked by formulæ 15, 17, 18 and 19 of the Code, the following result is obtained.

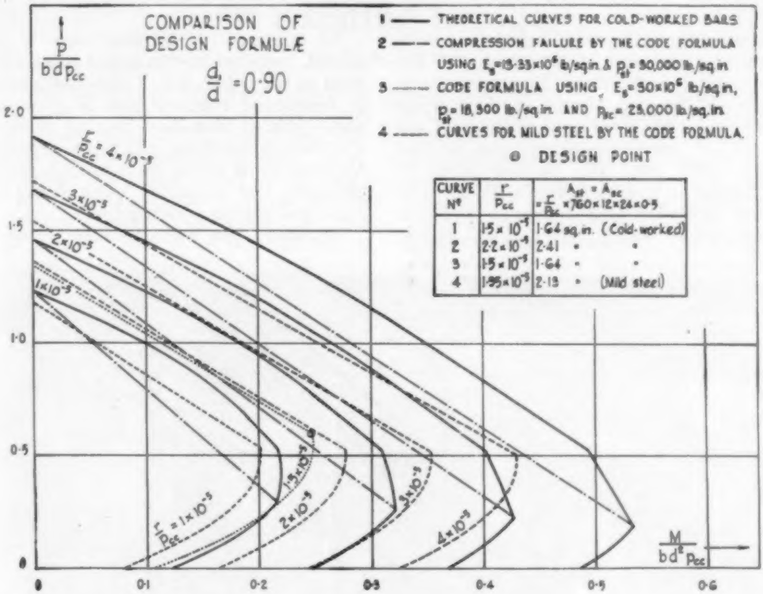
Column 24 in. × 12 in.; concrete 1 : 2 : 4; $p_{cc} = 760$ lb. per square inch; reinforcement $A_{sc} = A_{st} = 1{\cdot}64$ sq. in.; $\frac{d_1}{d} = 0{\cdot}90$, from which $d_1 = 21{\cdot}6$ in.; $e = 10$ in.; $p_{sc} = 23{,}000$ lb. per square inch; $p_{st} = 30{,}000$ lb. per square inch; $E_s = 13{\cdot}33 \times 10^6$ lb. per square inch.

From these values and the formulæ in the Code, $X = 0{\cdot}3617$; $P_b = 59{,}770$ lb.; $e_b = 23{\cdot}61$ in.; $P_o = 291{,}900$ lb.; $P = 110{,}360$ lb.; $M = 1{,}103{,}600$ in.-lb.

EXAMPLE 3.—The graph used in Example 1 also gives the stress in the tensile steel as 18,300 lb. per square inch. If this is used as the allowable steel stress in the Code formulæ then $p_{st} = 18{,}300$ lb. per square inch; $E_s = 30 \times 10^6$ lb. per square inch; $X = 0{\cdot}622$; $P_b = 130{,}000$ lb.; $e_b = 10$ in.; $P_o = 291{,}900$ lb.; $P = 130{,}000$ lb.; $M = 1{,}300{,}000$ in.-lb.

EXAMPLE 4.—If the same column is designed by the Code formulæ using 1·64 sq. in. of mild steel on each face, then $e = 10$ in.; $p_{sc} = 18{,}000$ lb. per square inch; $p_{sc} = 20{,}000$ lb. per square inch; $X = 0{\cdot}6071$; $P_b = 116{,}310$ lb.; $e_b = 10{\cdot}75$ in.; $P_o = 275{,}500$ lb.; $P = 121{,}200$ lb.; $M = 1{,}212{,}000$ in.-lb.

GRAPH No. II.



A comparison of Examples 2 and 4 shows that, in the case of similar columns reinforced with equal areas of cold-worked bars and mild steel, the Code formulæ allow a greater load on the column with mild steel reinforcement. This anomaly is due to the fact that the Code formulæ for compressive failure depend to a certain extent on the permissible tensile stress in the reinforcement, and this stress is not necessarily reached when compressive failure occurs.

A comparison of Examples 1 and 3 shows that the Code formulæ will give the same result as the graphs if the actual tensile stress in the steel is substituted for the permissible stress. *Graph* 2 gives a comparison of the results obtained in these four examples.

International Conditions of Contract.

AGREEMENT has been reached on a standard form of contract for civil engineering works which are the subject of international tenders. The documents have been prepared by the Fédération Internationale du Bâtiment et des Travaux Publics and the Fédération Internationale des Ingénieurs-Conseils and are recommended by these bodies for general use. The conditions conform in general to normal British practice. Copies may be obtained from the Export Group for the Constructional Industries, 21 Tothill Street, London, S.W.1, and the Association of Consulting Engineers, 36 Victoria Street, London, S.W.1; the price is 5*s*.

A Tall Building in London.

THE structures illustrated in *Fig.* 1 were started at the end of May last and are due to be completed by December 1958. There will be two groups of three office buildings, with a total of about 325,000 sq. ft. of accommodation and several shops. The two sites occupy most of Eastbourne Terrace, London, W.2. The tallest structure will comprise eighteen stories and be 200 ft. high; it will be one of the tallest inhabited buildings in London.

The cast-in-situ reinforced concrete frames will be clad with precast slabs faced with exposed Scottish granite aggregate. The infilling panels will generally be of 2-in. yellow bricks in the case of the four lower buildings and of glass mosaic on the two taller structures. Glass mosaics will also be used at street level on all the buildings. The main staircases, the floors of the entrance halls, the interiors of the cloakrooms, and the window sills will be finished in polished precast terrazzo.

At one end of the site there is good gravel, but cast-in-situ bored piles will be used at the other end. The frame will be of reinforced concrete cast in place with some precast non-load-bearing mullions. The tallest building will have reinforced concrete walls at the ends and around the lift-wells and staircase. The floors will be of hollow-tile construction.

Generally the walls will be of cavity construction with a brick outer leaf and an inner leaf of insulating blocks for the lower buildings; in the case of the towers there will be an outer leaf of precast concrete slabs faced with glass mosaic and an insulating inner leaf. The end walls of the towers will be 1 ft. 2 in. and 1 ft. thick, with silver-grey bricks. Above 50 ft. the inner faces of the walls will be finished with 2-in. of wood-wool slabs and plastered.

The architects are Messrs. C. H. Elsom & Partners, the consulting engineers Messrs. Clarke, Nichols & Marcel, and the contractors Tersons, Ltd.

Fig. 1.

Prestressed Footbridge Cantilevered from an Existing Bridge.

By N. A. DEWS.

THE road bridge over the river Wharfe at Otley is 21 ft. wide between parapets, and until the building of the new footbridge provided a carriageway of only 12 ft. The bridge comprises seven stone arches of varying spans totalling about 260 ft. The new footbridge (*Fig.* 1) has five spans of 38 ft. 3 in., one of 35 ft., and one of 31 ft., and is supported on balanced cantilevers projecting approximately over the centres of the piers.

The cantilevers are formed of two 16-in. by 6-in. rolled steel joists weighing 50 lb. per foot at 2 ft. 3 in. centres encased in concrete to dimensions of 3 ft. 9 in. wide by 1 ft. 9½ in. deep passing under the roadway and balanced by concrete blocks 8 ft. 6 in. long by 2 ft. 6 in. thick and about 8 ft. deep. The joists rest directly on a steel bearing 10½ in. inside the parapet, and the forward ends of the cantilevers are separated from the piers by jointing material ½ in. thick.

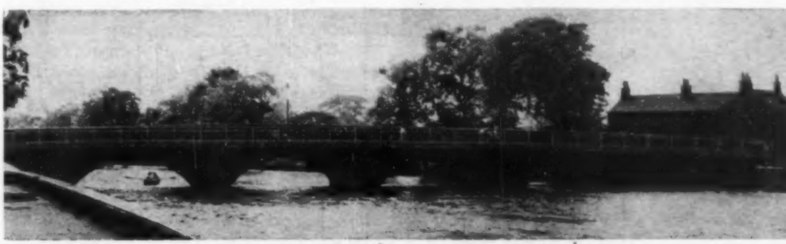


Fig. 1.—Cantilevered Footpath to Existing Bridge.

The footbridge is formed of prestressed precast beams, four in each span at 2 ft. 1 in. centres, connected at intervals of 6 ft. 4½ in. by three bolts of ⅝ in. diameter. Around these bolts small concrete diaphragms were cast in place to support the prestressed deck planks, 2 in. thick, which extend parallel to the beams; the bolts also provide a fixing for the tubular steel fence standards. With this arrangement the full width of the flanges of the beams is available for use (*Fig.* 2).

The prestressed beams and planks were made by Pre-Fact Concrete Co., Ltd., on the "long-line" system, using a 1 : 1 : 2 concrete with quartzite aggregate and rapid-hardening Portland cement. The concrete had a cube strength of about 8000 lb. per square inch at 28 days. A constant temperature of 60 deg. Fahr. was maintained during casting and curing, using steam, and after three to four days, when a cube strength of 4000 lb. per square inch was obtained, the tensioned wires were cut and the prestress applied. The centres of the beams lifted about ¾ in. when the prestress was applied; this was neutralised when the beams were tested under full working load, and with 50 per cent. overload there was a downward deflection of about ¼ in. with no observed residual deflection on removal of the load.

The maximum compressive stress in the concrete before the live load is applied is about 1300 lb. per square inch in both flanges, and under the full live load of 100 lb. per square foot the maximum stresses are 2400 lb. per square inch compression in the top and about 30 lb. per square inch tension in the bottom, assuming normal losses of prestressing force.

There are thirty-four 0·2-in. wires in each beam, tensioned initially to 120,000 lb. per square inch. The design allows for the beams being lifted at the centre in a cradle 6 ft. long without appreciable tension in the top flange. (The design and details of the prototype of this bridge were described in this journal for May, 1953.)

Short lengths of bridge rail are bolted to the steel joists in the cantilevers, and these rails are in contact with the steel bearing-plates cast in the beams and

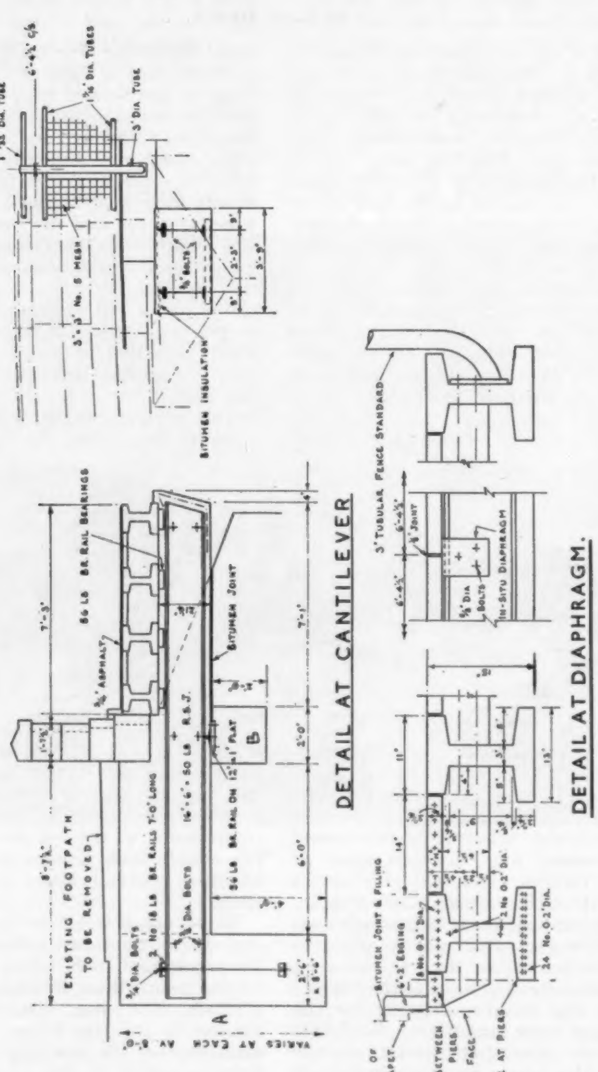


Fig. 2.—Details of Footbridge.

flush with their undersides, which are in turn separated from the cantilevered beam by a layer of bituminous material.

The fencing standards are 3-in. diameter tubes welded to a baseplate which engages with the bolts in the diaphragms. On the tops of the standards are plates $\frac{5}{16}$ in. thick with two set-screws that secure the top rails. Small sleeves welded in the standards support the two lower rails (*Fig.* 2), and $\frac{1}{4}$-in. riveted pins pass through the sleeves and tubes. The length of these is arranged so that any panel can be removed without disturbing the standards.

Some difficulty was expected in forming the part of the cantilevers under the deck of the bridge owing to the restricted width of roadway, which could not reasonably be closed for any appreciable time, and to the presence of six cables or pipes under the existing footpath. For this reason this part of the cantilever is short, so that the excavation for the kentledge blocks could be taken out without disturbing the use of the bridge. The six blocks (A in *Fig.* 2) were excavated for and concreted as a first operation, a square hole being left to accommodate the steel joists to be placed later. At the same time the spandrel wall over the piers was cut through, leaving the parapet undisturbed, and part of the excavation under the footpath was taken out through this hole from the river side of the wall.

Fig. 3.—Placing a Cantilever Girder.

Fig. 4.—Placing a Prestressed Beam.

The bearing blocks (B in *Fig.* 2) were then concreted and at the same time the remainder of the excavation across the footpath was removed and the gap bridged with timber and steel plates, causing little inconvenience to pedestrians.

All the steel joists were then slotted into position from the river side in one operation on a Sunday morning (*Fig.* 3), and the rear part of the cantilevers and the remainder of the kentledge blocks concreted, thus restoring the footpath and enabling the rest of the work to proceed undisturbed from the river side of the bridge. Fourteen prestressed beams were fixed between midnight and 6 a.m. before beginning to concrete the cantilevers (*Fig.* 4).

Fittings had been welded to the joists to support the shuttering for the cantilevers; this was easily erected from a platform suspended from the precast beams, the concrete being transported along the beams. High-alumina cement was used for the concrete in the cantilevers and diaphragms to enable the work

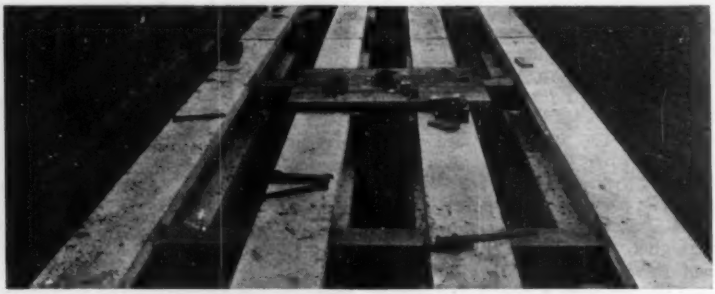


Fig. 5.—Diaphragms in Course of Construction.

to proceed quickly from one end to the other.

Timber shutters wedged from the bottom flanges of the beams were provided for all the diaphragms in one span, and these were stripped and refixed and the diaphragms concreted in one further span each day (*Fig.* 5). The deck planks were placed in the outer bay and the handrail fixed as the cantilevers and diaphragms were being completed, and several spans on the south side were finished before the last cantilever on the north end was concreted.

Telephone and electricity cables were then transferred to the footbridge; U-shaped brackets resting on the bottom flange supported hardboard in the inner bay on which the electricity cables were laid, and in the second bay telephone cables were supported on hooked hangers bolted to the diaphragms.

The cost of the work was about £6200; of this sum alterations to the public supply services cost £1100, the cantilevers and end abutments, etc., cost about £3100, leaving £2000 for the bridge, including fencing and surfacing. The surface of the footbridge is cold asphalt ¾ in. thick, and the inner side of the footpath is finished with a 6-in. by 2-in. concrete kerb bedded and jointed in cement mortar but backed with cold asphalt sealed with jointing material.

The bridge was completed by direct labour in twelve weeks and the greater part of this time was taken in the construction of the kentledge blocks and the embedded part of the cantilevers, and cutting through the spandrel walls. The work was carried out under the supervision of the West Riding County Engineer and Surveyor, Colonel S. Maynard Lovell, O.B.E., T.D., M.I.C.E., whose permission to publish this article is acknowledged.

A Church in Trinidad.

The illustration shows the church of St. Stephen at Prince's Town, Trinidad, the construction of which has recently been completed. The main building is 105 ft. 6 in. long, 30 ft. wide, and 31 ft. high from the floor to the ridge of the roof. The campanile is 54 ft. 6 in. high and 10 ft. by 6 ft. in plan. The walls are of hollow clay tiles supported by reinforced concrete beams and columns and rendered on both sides. The roof is of timber supported on timber trusses and covered with galvanised iron sheets.

The architect is Mr. J. Leslie and the consulting engineer Mr. R. D. Archibald, both of Trinidad. The work was done by a local builder with the aid of a small concrete mixer and a block and tackle.

Frames with Curved Beams.

By A. CHRONOWICZ.

THERE is an increasing use of curved beams as stiffening members in "shell" roofs. Their analysis must be considered in conjunction with straight beams and columns, and can be carried out conveniently to a predetermined degree of accuracy by the moment-distribution method. The elastic constants and the forces at the fixed ends for any loading can be calculated by the column-analogy method.

The usual simplifications of the analysis due to actual or assumed symmetry can be made as in the case of horizontal continuous beams. The theory is given for prismatic members, but it can easily be extended to beams with varying moments of inertia.

The stiffness S and carry-over factor r for a symmetrical arched beam (*Fig.* 1) are given by

$$S = \left(\frac{1}{A} + \frac{y_A{}^2}{I_x} + \frac{\tfrac{1}{4}l^2}{I_y}\right)\frac{I_o}{ds}, \quad r = \frac{-\dfrac{1}{A} - \dfrac{y_A{}^2}{I_x} + \dfrac{\tfrac{1}{4}l^2}{I_y}}{-\dfrac{1}{A} - \dfrac{y_A{}^2}{I_x} - \dfrac{\tfrac{1}{4}l^2}{I_y}}$$

in which A is the area, I_x and I_y the moments of inertia about axes X and Y of the analogous column, y_A the co-ordinate of the elastic centroid, l the span, I_o the moment of inertia of the cross section, and ds the linear increment of the arc. Moment M at the end of a curved beam produces horizontal and vertical forces V and H which may be evaluated in terms of M and other elastic constants of the analogous column by equating the end displacements to zero (*Fig.* 2). If $M_\xi = M - V\xi - H\eta$, the equations are

$$-M\int\frac{ds}{EI}\xi + V\int\frac{ds}{EI}\xi^2 + H\int\frac{ds}{EI}\xi\eta = 0$$

$$-M\int\frac{ds}{EI}\eta + V\int\frac{ds}{EI}\xi\eta + H\int\frac{ds}{EI}\eta^2 = 0$$

or, introducing the usual column-analogy constants,

$$VI_\eta + HI_{\xi\eta} = MA\bar{\xi}\,; \quad VI_{\xi\eta} + HI_\xi = MA\bar{\eta}.$$

Similarly, the fixed-end moments and forces at the ends due to a horizontal displacement Δ can be found from the strain-energy equations (*Fig.* 3).

$$MA + HA\bar{\eta} = 0\,; \; MA\bar{\eta} + HI_\xi = \Delta$$

From these, $H = -\dfrac{M}{\bar{\eta}}$ and

$$M = \frac{\Delta}{A\bar{\eta} - \dfrac{I_\xi}{\bar{\eta}}} = \frac{\Delta}{A\bar{\eta} - \dfrac{1}{\bar{\eta}}[I_x + A(\bar{\eta})^2]}$$

which, when transferred to the centroidal co-ordinates, becomes $M = -\dfrac{\Delta\bar{\eta}}{I_x}$.

Similarly, the moments due to vertical displacement Δ are (*Fig.* 4)

$$M_A = \frac{\Delta\,.\,\tfrac{1}{2}l}{I_y}, \quad M_B = -\frac{\Delta\,.\,\tfrac{1}{2}l}{I_y}, \quad V = \pm\frac{\Delta}{I_y}.$$

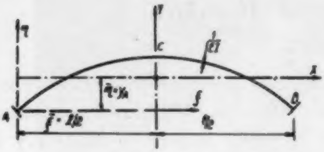
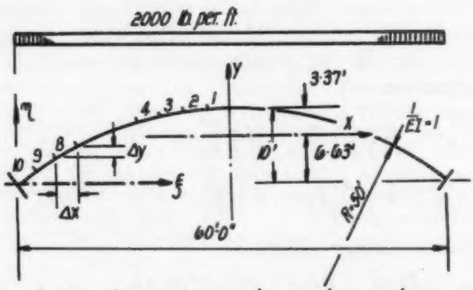


Fig. 1.

Fig. 2.

Fig. 3.

Fig. 4.



$s = 64{\cdot}34'$

$\Delta s = 1/20\, s = 3{\cdot}217'$

$\Sigma\, \eta = 66{\cdot}28$

$\bar{\eta} = 6{\cdot}628'$

$\Sigma\, y = 0{\cdot}01$

$M^{\circ} = 900{,}000$ lb. ft

$\Sigma x^2 + 1/12\, \Sigma (\Delta x)^2 = 3176{\cdot}0$

$\Sigma y^2 + 1/12\, \Sigma (\Delta y)^2 = 89{\cdot}8$

$1/2\, Gi = 5832$

$1/2\, Mxx = 8121$

$1/2\, A = 10$

(Moments tabulated in 1000 lb. ft units)

	x	Δx	η	y	Δy	Ms
1	1·60	3·21	9·973	3·35	0·10	897·5
2	4·82	3·20	9·766	3·14	0·31	876·8
3	8·01	3·18	9·353	2·73	0·52	835·8
4	11·16	3·14	8·737	2·11	0·72	775·4
5	14·27	3·08	7·919	1·29	0·92	696·3
6	17·33	3·01	6·901	0·27	1·11	599·7
7	20·30	2·94	5·691	−0·94	1·31	487·9
8	23·20	2·85	4·289	−2·34	1·49	361·8
9	26·00	2·75	2·707	−3·92	1·68	224·0
10	28·70	2·63	0·944	−5·68	1·84	76·3

Fig. 5.

In these expressions $\frac{1}{2}l$ and $\bar{\eta}$ are the co-ordinates of the elastic centroid, $I_{\xi\eta}$ the product of inertia in the ξ, η system, and I_x, I_y the moments of inertia about the centroidal axes.

In order not to obscure the principle, using an arch that is a segment of a circle with a radius of 50 ft., and a span of 60 ft., the method is illustrated by simple numerical examples, forming part of a continuous frame.

The elastic properties of the arched rib are given in *Fig.* 5; loads are in units of 1000 lb. and moments in units of 1000 lb.-ft.

The thrust and fixed-end moments due to a uniformly-distributed load of 2000 lb. are

$$H = \frac{8121 \times 10^3}{89 \cdot 8} = 90 \cdot 43 \times 10^3 \text{ lb.}$$

$$M_F = -\frac{5832 \times 10^3}{10} + 90 \cdot 43 \times 10^3 \times 6 \cdot 63 = 16 \cdot 2 \times 10^3 \text{ lb.-ft.}$$

The stiffness of the rib is given by

$$\frac{\Delta S}{I_0} S = \frac{1}{10} + \frac{6 \cdot 628^2}{89 \cdot 8} + \frac{30 \cdot 0^2}{3176 \cdot 0} = 0 \cdot 873,$$

the carry-over factor is $r = -\dfrac{0 \cdot 306}{0 \cdot 873} = -0 \cdot 35$, and the fixed-end moment due to horizontal displacement Δ is $M_\Delta = \dfrac{6 \cdot 628\Delta}{179 \cdot 6} = 0 \cdot 0369\Delta$.

The factors of the moment of inertia in the (ξ, η) system are

$\frac{1}{2}I_\eta = 3176 \cdot 0 + 10 \times 30 \cdot 0^2 = 12{,}176$; $\frac{1}{2}I_\xi = 89 \cdot 8 + 10 \times 6 \cdot 63^2 = 529 \cdot 1$;
$\frac{1}{2}I_{\xi\eta} = 10 \times 30 \cdot 0 \times 6 \cdot 63 = 1988 \cdot 4$.

The static moments are, $\frac{1}{2}A\bar{\xi} = 300$ and $\frac{1}{2}A\bar{\eta} = 66 \cdot 28$.

From the two simultaneous equations

$$\begin{pmatrix} 12{,}176 \cdot 0 & 1988 \cdot 4 \\ 1988 \cdot 4 & 529 \cdot 1 \end{pmatrix} \begin{pmatrix} V \\ H \end{pmatrix} = \begin{pmatrix} 300 \cdot 0 \\ 66 \cdot 28 \end{pmatrix} M$$

the forces V and H at the ends due to the applied moment M are

$$V = 0 \cdot 0108M; \quad H = 0 \cdot 0847M.$$

EXAMPLE I.—Analyse the symmetrical frame comprising a curved beam supported on columns 16·09 ft. high (*Fig.* 6).

For the analogy with the curved rib, the columns are divided into five elements ΔS of 3·217 ft. Their elastic constants are:

$$A = 5; \quad r = 0 \cdot 5; \quad \Sigma y^2 + \tfrac{1}{12}\Sigma(\Delta y)^2 = 107 \cdot 7.$$

$$\frac{\Delta S}{I_0} S = \frac{1}{5} + \frac{8 \cdot 043^2}{107 \cdot 7} = 0 \cdot 801; \quad M_\Delta = \frac{8 \cdot 043\Delta}{107 \cdot 7} = 0 \cdot 0747\Delta.$$

The moment-distribution factors at the joint B are

$$\alpha_{BA} = \frac{0 \cdot 801}{1 \cdot 674} = 0 \cdot 478; \quad \alpha_{BD} = 0 \cdot 522.$$

It is first assumed that there is no horizontal displacement of the joints B and D, and the moment distribution is carried out (*Fig.* 7).

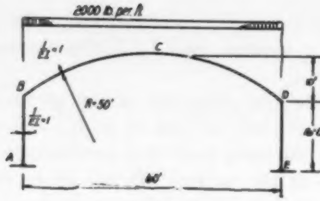


Fig. 6.

AB	BA	BD	(−0·35)
	0·478	0·522	
		16·2	
−3·86	−7·73	−8·47	
		−2·96	
0·70	1·41	1·55	
		0·54	
−0·13	−0·26	−0·28	
		−0·10	
0·02	0·05	0·05	
		0·02	
	−0·01	−0·01	
−3·3	−6·5	6·5	

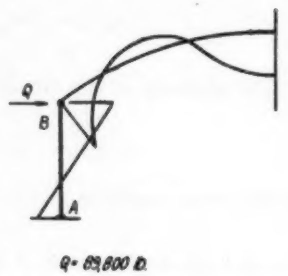
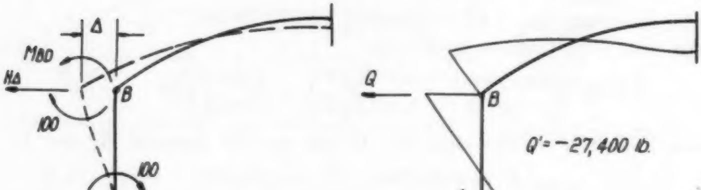


Fig. 7.



AB	BA	BΔ	(−0·35)
	0·478	0·522	
100·0	100·0	−98·8	
−0·29	0·58	−0·62	
		−0·22	
0·05	0·10	0·12	
		0·4	
−0·01	−0·02	−0·02	
99·8	99·5	−99·5	

Fig. 8.

The propping reaction $Q = 89{,}800$ lb. is composed of (1) the static reaction of the fixed-end arch, 90,430 lb.; (2) the elastic reaction of the columns $\frac{(3\cdot3 + 6\cdot5) \times 10^3}{16\cdot09} = 610$ lb., and (3) the reaction due to balancing moments applied at end of the rib BD, $-7\cdot2 \times 0\cdot085 \times 2 \times 10^3 = -1220$ lb. *Fig.* 8 shows the moments and forces due to a displacement Δ, producing a fixed-end moment of 100,000 lb.-ft. at the ends of the column AB.

The resulting moment on the curved rib is

$$M_{BD}{}^{\Delta} = -2 \times 100 \times \frac{0\cdot0369}{0\cdot0747} \times 10^3 = -98{,}800 \text{ lb.-ft.},$$

and the force required to produce this displacement of the rib is

$$H_{BC} = -\frac{98\cdot8 \times 10^3}{6\cdot63} = -14{,}910 \text{ lb.}$$

The total force Q' at springings B and D is then

$$Q' = \left\{-14\cdot91 - (99\cdot8 + 99\cdot5)\frac{1}{16\cdot09} - 2 \times 0\cdot52 \times 0\cdot085\right\} \times 10^3 = -27{,}400 \text{ lb.}$$

The sway moments already found must therefore be multiplied by the correcting factor $\alpha = 89\cdot8/27\cdot4 = 3\cdot277$ and added to the moments under propped conditions. The final moments are shown in *Fig.* 9. The thrust at the bottom of the column is

$$H_A = (324 + 320) \times \frac{10^3}{16\cdot09} = 40{,}000 \text{ lb.}$$

The symmetry of the structure can be used by introducing the end-rotation factor

$$k = \frac{1 - r_a.r_b}{1 - r_a\dfrac{M_B}{M_A}} = \frac{1 - (-0\cdot35)^2}{1 - (-0\cdot35)(-1)} = 1\cdot35.$$

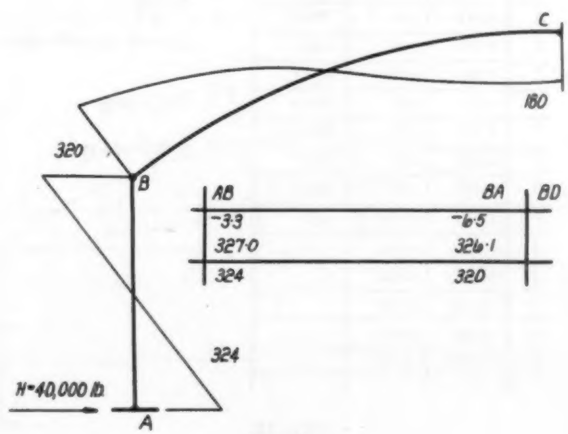


Fig. 9.

AB	BA	BD
	0·405	0·595

Propped condition.

AB	BA	BD
		16·2
−3·3	−6·6	−9·6
−3·3	−6·6	6·6

Arbitrary sway.

AB	BA	BD
100·0	100·0	98·8
−0·3	−0·6	−0·6
99·7	99·4	−99·4

Final moments

	AB	BA	BD
	−3·3	−6·6	
	326·7	325·7	
Σ	323	319	

$$Q = \cdots\cdots\cdots \; 90{\cdot}43$$

$$\frac{6{\cdot}6 + 3{\cdot}3}{16{\cdot}09} = 0{\cdot}67$$

$$-2 \times 9{\cdot}6 \times 0{\cdot}085 \; \frac{0{\cdot}87}{1{\cdot}18} \quad -1{\cdot}21$$

$$89{\cdot}8$$

$$Q' = -27{,}400 \text{ lb}$$

$$\alpha = 3{\cdot}277$$

$$H = \frac{323 + 319}{16{\cdot}09} = 39{,}900 \text{ lb}$$

Fig. 10.

	η	y	Ms
1	9·973	8·24	897·5
2	9·766	8·03	876·8
3	9·353	7·62	835·8
4	8·737	7·00	775·4
5	7·919	6·78	696·3
6	6·901	5·16	599·7
7	5·691	3·95	487·9
8	4·289	2·55	361·8
9	2·707	0·97	224·0
10	0·944	−0·79	76·3
11	−1·609	−3·35	
12	−4·826	−6·56	
13	−8·043	−9·78	
14	−11·260	−13·00	
15	−14·477	−16·22	

$$\Sigma\eta = 26{\cdot}065$$

$$\bar{\eta} = -1{\cdot}738$$

$$\tfrac{1}{2}A = 15$$

$$\Sigma y^2 + \tfrac{1}{2}\Sigma(\Delta y)^2 = 315{\cdot}3$$

$$\tfrac{1}{2}G_i = 5832$$

$$\tfrac{1}{2}Mx = 36637$$

Fig. 11.

The modified stiffness of rib BD is $\frac{\Delta S}{I_o} S'_{BD} = 0{\cdot}873 \times 1{\cdot}35 = 1{\cdot}179$,

and the moment-distribution factors at joint B are $\alpha_{BA} = 0{\cdot}405$, and $\alpha_{BD} = 0{\cdot}595$.

In *Fig.* 10 the simplified moment-distribution operations for propped conditions and for sway are tabulated.

The horizontal force at the foot of the frame is calculated by the direct application of the column analogy, shown in *Fig.* 11,

$$H = \frac{36{,}637 \times 10^3}{915{\cdot}3} = 40{,}000 \text{ lb.}$$

$$M_A = \left(-\frac{5832}{15} + 40 \times 17{\cdot}82\right) \times 10^3 = 325{,}000 \text{ lb.-ft.}$$

$$M_B = \left(-\frac{5832}{15} + 40 \times 1{\cdot}74\right) \times 10^3 = -\ 319{,}000 \text{ lb.-ft.}$$

$$M_C = \left(900 - \frac{5832}{15} - 40 \times 8{\cdot}26\right) \times 10^3 = 181{,}000 \text{ lb.-ft.}$$

EXAMPLE II.—*Fig.* 13 shows the analysis of the unsymmetrical frame, which may also be solved by the direct application of column analogy (*Fig.* 12). Here, however, the computations are involved and not easy to verify. The co-ordinates of the centroid S are given by

$$\bar{\xi} = \frac{20 \times 30}{25} = 24 \text{ ft.}; \qquad \bar{\eta} = (20 \times 6{\cdot}63 - 5 \times 8{\cdot}045)\frac{1}{25} = 3{\cdot}695 \text{ ft.}$$

while the point of application G of the analogous load is

$$x_G = 6 \text{ ft.},\ y_G = \frac{8121}{5832} + 6{\cdot}628 - 3{\cdot}695 = 4{\cdot}32 \text{ ft.}$$

The moment of inertia factors with respect to axes X, Y are

$$I_X = 2 \times 89{\cdot}8 + 20 \times 2{\cdot}93^2 + 107{\cdot}7 + 5 \times 11{\cdot}74^2 = 1148$$
$$I_Y = 2 \times 3176 + 20 \times 6^2 + 5 \times 24^2 = 9952$$
$$I_{XY} = 20 \times 2{\cdot}93 \times 6 + 5 \times 11{\cdot}74 \times 24 = 1760$$

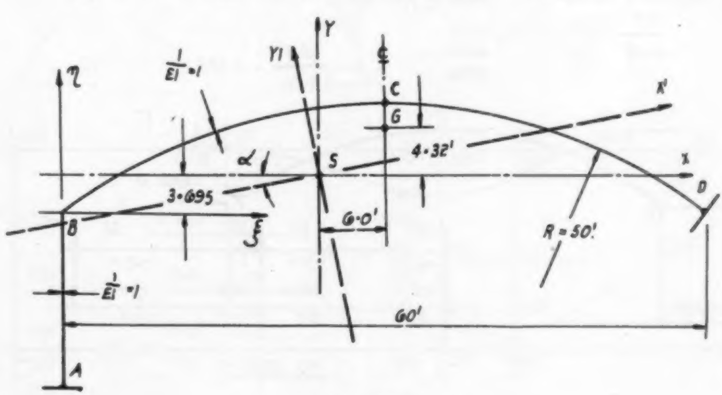


Fig. 12.

The inclination of the principal axes X′ and Y′ of the section is found from

$$\tan 2\alpha = \frac{2 \times 1760}{9952 - 1148} = 0{\cdot}3998, \text{ from which } \alpha = 10^\circ\ 54',$$

and the principal moments of inertia are found from

$$I' = \tfrac{1}{2}\Sigma I \pm \tfrac{1}{2}\Delta I \sec 2\alpha,\ I_X' = 807,\ I_Y' = 10{,}293.$$

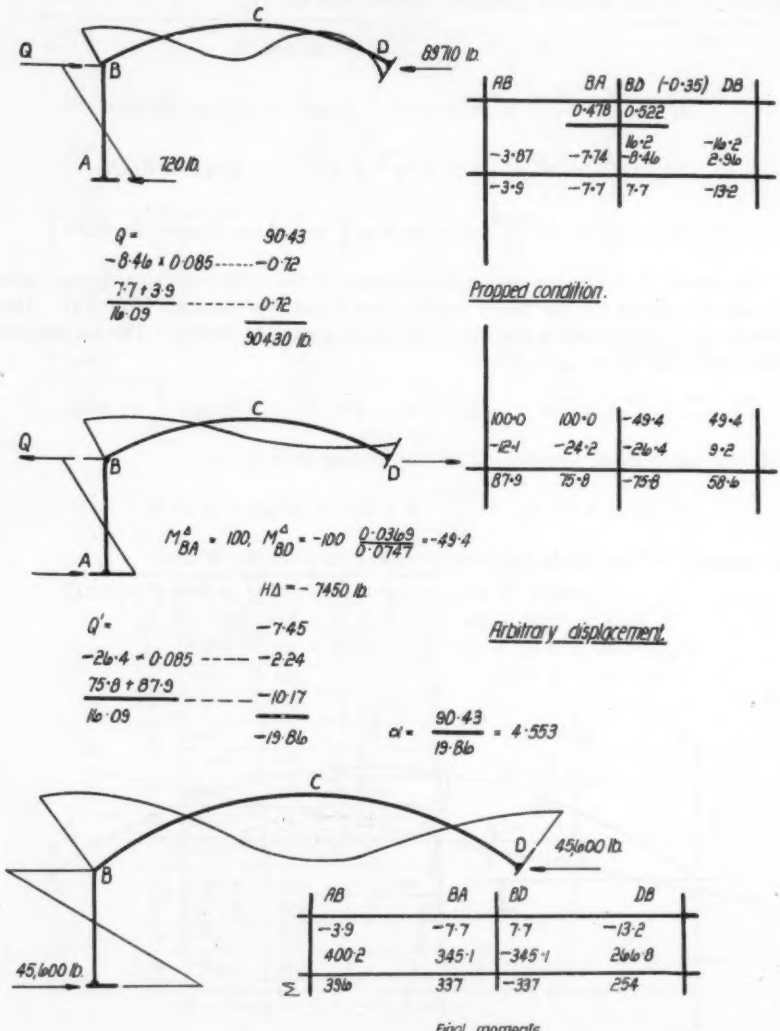


Fig. 13.

The new co-ordinates x', y', which can be calculated from

$$x' = y \sin \alpha + x \cos \alpha, \quad y' = y \cos \alpha - x \sin \alpha, \text{ are}$$

$$A(-27{\cdot}31,\ -14{\cdot}90),\ B(-24{\cdot}27,\ 0{\cdot}91),\ D(34{\cdot}65,\ -10{\cdot}44),\ G(6{\cdot}71,\ 3{\cdot}11).$$

The moments of the analogous load about the centroidal axes are

$$M_{X}' = 11{,}664 \times 3{\cdot}11 \times 10^3 = 36{,}275 \times 10^3 \text{ lb.-ft.}$$
$$M_{Y}' = 11{,}664 \times 6{\cdot}71 \times 10^3 = 78{,}265 \times 10^3 \text{ lb.-ft.}$$

The corresponding elastic reactions are

$$H' = \frac{36{,}275 \times 10^3}{807} = 44{,}950 \text{ lb.} \qquad \Delta V' = \pm \frac{78{,}265 \times 10^3}{10{,}293} = \pm 7600 \text{ lb.}$$

and the final moments are

$$M_A = \left(-\frac{11{,}664}{25} + 44{\cdot}95 \times 14{\cdot}9 + 7{\cdot}60 \times 27{\cdot}31\right) \times 10^3 = 411{,}000 \text{ lb.-ft.}$$

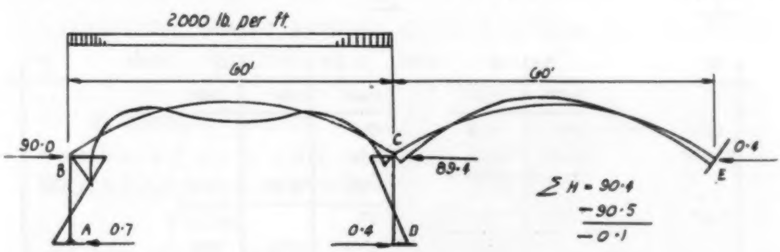
$$M_B = \left(-\frac{11{,}664}{25} - 44{\cdot}95 \times 0{\cdot}91 + 7{\cdot}6 \times 24{\cdot}27\right) \times 10^3 = -323{,}000 \text{ lb.-ft.}$$

$$M_D = \left(-\frac{11{,}664}{25} + 44{\cdot}95 \times 10{\cdot}44 - 7{\cdot}6 \times 34{\cdot}65\right) \times 10^3 = -261{,}000 \text{ lb.-ft.}$$

The horizontal and vertical loads are

$$H = (44{\cdot}95 \times 0{\cdot}982 + 7{\cdot}6 \times 0{\cdot}1891) \times 10^3 = 45{,}600 \text{ lb.}$$
$$V_A = (60 + 44{\cdot}95 \times 0{\cdot}1891 - 7{\cdot}6 \times 0{\cdot}982) \times 10^3 = 61{,}000 \text{ lb.} \quad V_B = 59{,}000 \text{ lb.}$$

EXAMPLE III.—This illustrates the case of correcting for sway in two stages. *Fig.* 14 shows the moments and forces when propped. The propping forces at



AB	BA	BC (−0·35)	CB	CD	CE (−0·35)	EC
	0·478	0·522	0·343	0·314	0·343	
		16·2	−16·2			
−3·86	−7·73	−8·47	2·96			
		−1·59	4·54	4·16	4·54	−1·59
0·38	0·76	0·83	−0·29			
		−0·04	0·10	0·09	0·70	−0·04
0·01	0·02	0·02				
−3·5	−7·0	7·0	−8·9	4·3	4·6	−1·6

Fig. 14.

the joints B and C are $Q_B = 90{,}000$ lb., $Q_C = -89{,}400$ lb. In the first stage (*Fig.* 15) joint B is given an arbitrary displacement Δ. The corresponding fixed-end moments are calculated and distributed through the members of the frame, and the forces due to sway are calculated,

$$Q_B' = -18{,}100 \text{ lb.}; \quad Q_C' = 8300 \text{ lb.}$$

Fig. 16 shows the second stage with joint C displaced by Δ and joint B restrained against horizontal movement. The forces due to sway are

$$Q_B'' = -7800 \text{ lb.}; \quad Q_C'' = 25{,}400 \text{ lb.}$$

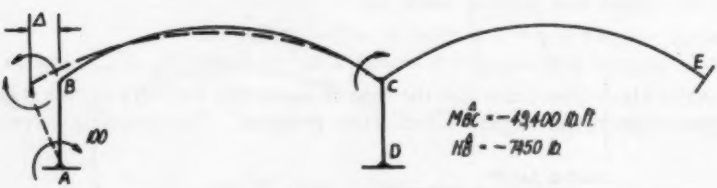
The correcting factors $\alpha = 4{\cdot}022$ and $\beta = 2{\cdot}205$ are calculated from

$$\begin{pmatrix} H_B \\ H_C \end{pmatrix} = \begin{pmatrix} 18{\cdot}1 & 7{\cdot}8 \\ 8{\cdot}3 & 25{\cdot}4 \end{pmatrix} \begin{pmatrix} \alpha \\ \beta \end{pmatrix} = \begin{pmatrix} 90{\cdot}0 \\ 89{\cdot}4 \end{pmatrix}.$$

The final moments obtained from $M = M^\circ + \alpha M' + \beta M''$ are shown in *Fig.* 17. The horizontal forces are calculated as follows:

$$H_A = (370 + 338)\frac{10^3}{16{\cdot}09} = 44{,}000 \text{ lb.}$$

$$H_D = -(285 + 253)\frac{10^3}{16{\cdot}09} = -33{,}400 \text{ lb.}$$



AB	BA	BC (−0·35)	CB	CD	CE (−0·35)	EC
	0·478	0·522	0·345	0·314	0·343	
100·0	100·0	−49·4	49·4			
−12·09	−24·19	−26·41	9·24			
		7·04	−20·11	−18·42	−20·11	7·04
−1·68	−3·37	−3·67	1·28			
		0·15	−0·44	−0·40	−0·44	0·15
−0·03	−0·07	−0·08	0·03			
			−0·01	−0·01	−0·01	
86·2	72·4	−72·4	39·4	−18·8	−20·6	7·2

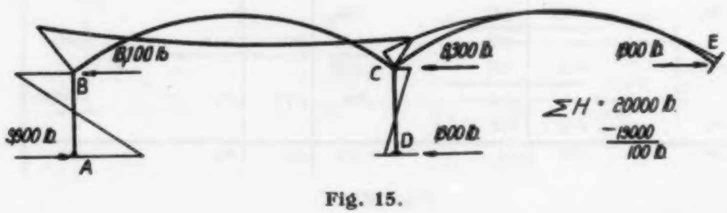


Fig. 15.

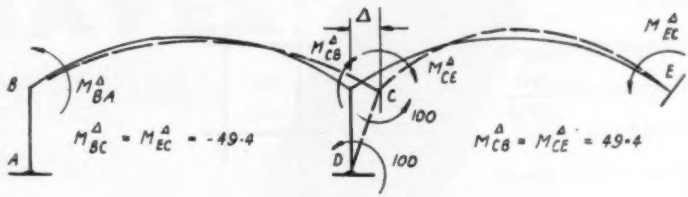
$$H_E = (-0{\cdot}4 + 1{\cdot}8 \times 4{\cdot}022 - 7{\cdot}8 \times 2{\cdot}205) \times 10^3 = -10{,}400 \text{ lb.}$$
$$\Sigma H = 44{,}000 - 43{,}800 = 200 \text{ lb.}$$

Fig. 18 shows the arrangement of props for a two-span arched rib stiffening a north-light shell. Here the complete analysis requires the solution of five simultaneous equations, while seven equations are needed for a three-span frame. However, for flat arches the fixed-end moments are very small and may be neglected in the preliminary analysis, and the thrust of the arch may be calculated from $H \simeq \frac{M}{d}$, in which M is the moment at midspan and d the rise of the arch.

In Example I, the approximate value of the thrust would be

$$H \simeq \frac{2 \times 60^2}{8 \times 10} \times 10^3 = 90{,}000 \text{ lb.,}$$

compared with 90,400 lb. by the exact solution.



AB	BA	BC	(-0·35)	CB	CD	CE	(-0·35	EC
	0·478	0·522		0·343	0·314	0·343		
		−49·4		49·4	−100·0	49·4		−49·4
11·80	23·61	25·79		−9·03				
		−1·23		3·51	3·21	3·51		−1·23
0·29	0·59	0·64		−0·22				
		−0·03		0·08	0·06	0·08		−0·03
	0·01	0·02						
12·1	24·2	−24·2		43·7	−96·7	53·0		−50·7

DC
−100·0
1·60
0·03
−98·4

$\Sigma H = 27{,}700$ lb.
$-27{,}700$
0

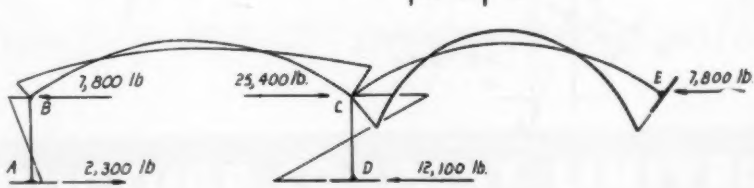
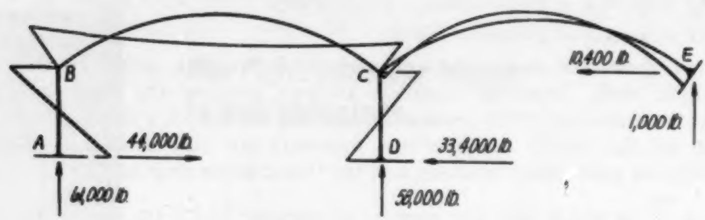


Fig. 16.



AB	BA	BC	CB	CD	CE	EC
−35	−7·0	7·0	−8·9	4·3	4·6	−1·6
346·7	291·2	−291·2	158·4	−75·6	−82·8	29·0
26·7	53·4	−53·4	96·4	−213·2	116·9	−111·8
370	338	−338	246	−285	3·9	−84

$$\sum H = 44{,}000 \text{ lb.} - 43{,}800 = 200 \text{ lb.}$$

DC
2·1
−37·8
−217·0
−253

Fig. 17.

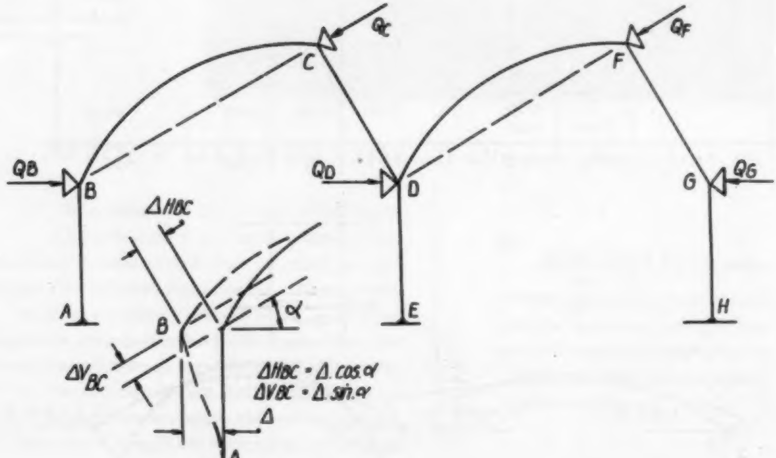


Fig. 18.

Assuming the point of contraflexure of the columns to be at a distance of one-third of the height from the base, $H \simeq \frac{900}{21} \times 10^3 = 43{,}000$ lb., which differs by only 7 per cent. from the actual value of 40,000 lb. and may be used either to check the analysis or in the preliminary design.

The last bay of the multiple-span frame may safely be assumed to be equivalent to the structure shown in *Fig.* 12. On this assumption the moments in *Figs.* 14 to 17 are

$$M_A = 396{,}000 \text{ lb.-ft.}, \quad M_B = 337{,}000 \text{ lb.-ft.}$$
$$M_C = 254{,}000 \text{ lb.-ft.}, \quad \text{and} \quad H = 45{,}600 \text{ lb.},$$

compared with 370,000 lb.-ft., 338,000 lb.-ft., 246,000 lb.-ft., and 44,000 lb., respectively, obtained by exact calculation. The calculation may be shortened by using the tables or graphs giving the elastic constants, fixed-end moments, and the reactions of the supports, which have been published.

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Scottish Hydro-electric Schemes.

A NEW project, known as the Awe Project, of the North of Scotland Hydro-electric Board, includes the first large pumped-storage hydro-electric development in Scotland. The scheme proposes to use the water from 324 square miles of the river Awe catchment area, and will be in three parts, namely the Inverawe, Cruachan, and Nant sections. The total capacity will be about 450,000 kW. and the estimated cost £24,500,000.

The Inverawe section includes the construction of a low barrage a short distance downstream from the outlet of Loch Awe, from which water would be taken by tunnel to a power station, with a capacity of about 35,000 kW., near the mouth of the river.

The Cruachan section includes the pumped-storage plant and will have a capacity of about 400,000 kW. The station will use, at night and at week-ends when the load is low, the surplus energy from generating stations for pumping water to a high level reservoir, which will be connected by shafts to an underground power station ; the water used will be conveyed to and from Loch Awe by a horizontal tunnel. The plant will be supplied mainly with water pumped from Loch Awe, supplemented by water taken by aqueducts from the headwaters of the rivers Liever and Noe.

The Nant section includes the raising of the level of Loch Nant by the construction of a dam near its outlet and conveying the water by tunnel and pipeline to a power station near Inverinan on the shore of Loch Awe. The station will have a capacity of 18,000 kW.

Another Proposed "Closed Shop" for Engineers.

DESPITE the unsuccessful attempt of the Durham County Council in 1952 to force the professional engineers on its staff to join a trade union, the Edmonton Borough Council is now making a similar attempt by adopting the following condition of employment: "That every employee, with the exception of doctors, nurses, and others in similar professions, shall during the term of his/her employment by the Edmonton Council be and continue to be a member of a trade union affiliated to, or specified by the Trades Union Congress as eligible for affiliation to, the Trades Union Congress; that every employee shall produce to the Town Clerk on request proof of membership of such trade union; and that failure to comply with this condition renders any employee liable to instant dismissal". The Town Clerk has since stated that the term "similar professions" is confined to dentists, health visitors, and midwives, and does not apply to professional engineers.

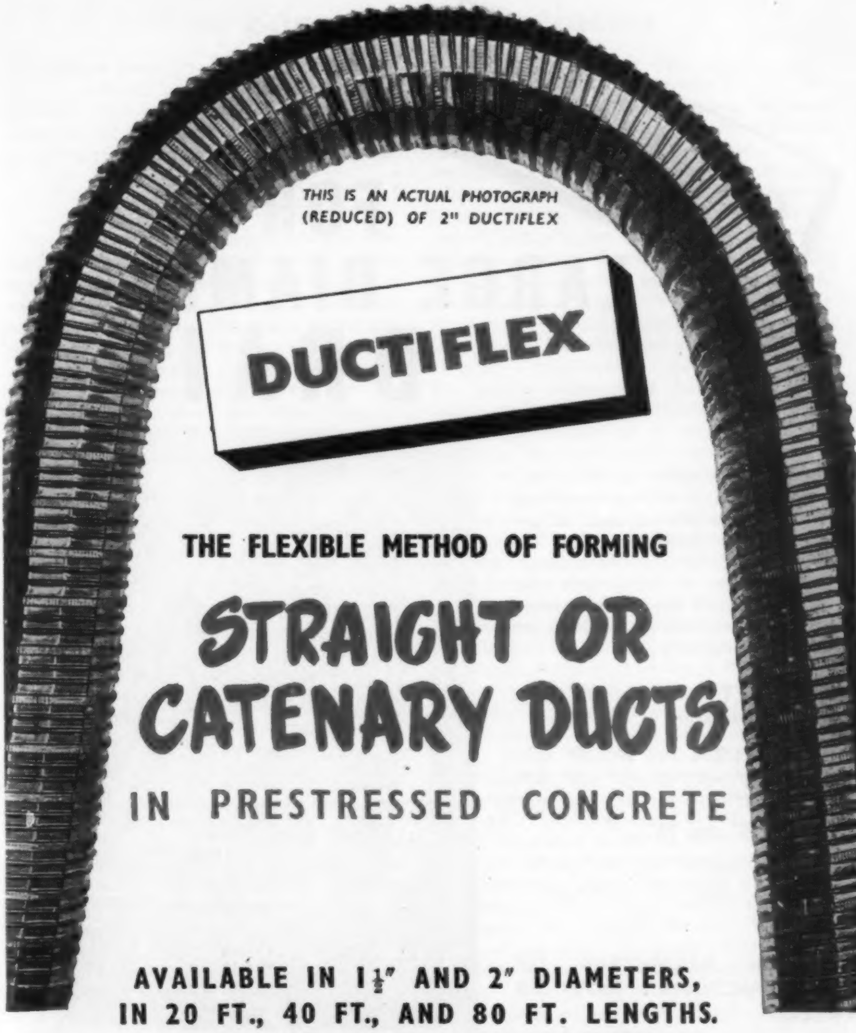
The Engineers' Guild is striving to have professional engineers exempted from this condition of employment, and it is to be hoped that it will be as successful as it was in the case of the Durham County Council. A remarkable feature of this case is that people concerned with health, who are the only members of the staff not subjected to this tyranny, are probably already members of a trade union, as may be the rest of the officials and staff from the Town Clerk downwards. The "closed shop" therefore seems to be an attempt to force the engineering staff to join trade unions catering for engineers or employees of local authorities.

Since a candidate's political views became his most important asset for election to a local council some queer antics have been performed by some queer and cranky councils. At Edmonton it is thought right that a medical officer of health and his staff need not belong to a trade union, but that the town clerk, the architect, the engineer, and all other members of the staff must show their cards. Why are these councillors afraid to interfere with anyone concerned with public health but anxious to enforce their political beliefs on everyone else on their pay-roll? Why may a doctor and a "health visitor" be allowed to retain their freedom while a borough engineer and a sanitary inspector must bow to the whim of a political tyrant? Is a trade union card of more importance than a man's ability in his profession? Surely it is clear that if a good and a bad candidate apply for a post, the bad candidate will be accepted if he has a union card and the good one has not—and surely this state of affairs is not in the best interests of the people who voted the councillors into power.

It may be suggested that the professional members of the staff of this council form their own closed shop—and preserve their freedom and self-respect—by resigning. The action of the council in attempting to force professional engineers to join a trade union is monstrous; the submission of the rest of the professional staff other than those concerned with "health services" to this tyranny is not easily understood.

Expanded-Shale Aggregate.

THE shrinkage on drying and expansion on wetting of expanded-shale aggregate are about 0·04 per cent., and not as stated in our July number.



The Education of Engineers.

An address given by Mr. R. V. Collman, B.Sc., A.M.I.C.E., to the Yorkshire Branch of the Engineers' Guild was published in the Journal of the Guild and is reprinted in the following by permission.

The Value of History.

Several universities and technical colleges are now experimenting with engineering curricula which include tuition in non-engineering subjects. Economics, law, philosophy, biology, social anthropology—all these, and others, are testimony of the growing endeavour to broaden the base of engineering education. In the following it is argued that the study of history—particularly technological history—has an especial value towards a fuller appreciation of the social implications of technical invention and development.

The unsatisfactory status of the engineering profession is a recurrent theme of discussion. What are the reasons, we ask, for the meagre rewards, for the lack of recognition of our profession, and for the failure of the general public to appreciate the part played by the engineer in the life of the modern community? All kinds of explanations have been given to such questions as these, but we have been reluctant to accept the obvious answer: that the community does not consider that the engineer deserves a higher status. The people clearly do not think that we are fitted for high administrative posts, and prefer to be directed and governed by politicians, lawyers and accountants. Let us then, for a moment, accept this view; let us forget what we conceive to be our "rights," and see what can be done to merit a higher status—not to command success, but to deserve it.

The engineer, as popularly envisaged, is a clever fellow who can do things if someone tells him what to do. He is seen essentially as a servant of the directing class. Politicians or financiers decide that it would be a good thing to have a bridge here, a road there, or a power station somewhere else. The engineer comes in only when he is called. In this view it is not engineers who guide the trend of progress, but the administrators.

This conception of the engineer's function in society has been strengthened in recent years by the mildly contemptuous label "back-room boy." We are even told that the engineer becomes so absorbed in his work that he is bound to become something of a recluse. And, indeed, the present trend of our training makes it inevitable that this should happen. Engineering graduates are being turned out who are only half educated in any real sense. Some, if we are to judge by their written work, are almost illiterate, fitted no doubt to become excellent back-room boys, but little more than that. The community, it would seem, is starting to develop "conditioned types," fitted only for narrow specialised functions. This is no new problem; indeed we are only too keenly aware of it, and many suggestions have been put forward on how engineers may become better fitted to be leaders of the community.

One school of reform has pinned its faith in action aimed at protecting or promoting the interests of the profession, such as the movement to establish the state registration of engineers. Such efforts may indeed be essential in winning full recognition for the profession, but they do not go to the root of the matter, which is to make engineers more worthy of the improved status that we all want.

The second line of attack, slower but surer, is to reform the education and training of the profession, so that engineers may develop a wider and more liberal outlook. The Engineers' Guild has done some excellent work in this direction; in London it has organised courses in management, public speaking, and other neglected fields. At our branch meetings, whenever we venture beyond the confines of our daily professional round, we are helping to open the windows a little wider. The movement has spread to engineering colleges; at Cambridge and some of the London colleges lectures are given on economics, law, and accountancy, and quite recently we have read that students at Loughborough College are pursuing a course in philosophy.

All these additions to an engineer's education are valuable, but they do not go deep enough; they are but the trimmings of culture. Far more effective would be a sustained pursuit of one of the broader humanities, and no subject is

better fitted to redeem our education than is the study of history—particularly technological history and its social implications.

Carlyle said that "man is a tool-using animal," and the history of civilisation has been determined by the progress of technology. Unfortunately, our arts-trained historians have not understood the key position of technology, while our technologists have been so immersed in present problems that they have disregarded the importance of history as a means of understanding the world around them. Our profession, if it were inspired by an historical consciousness, could make an invaluable contribution to the happiness and progress of man.

How History could be Studied.—We must begin with the student, and here a protest might be made against the too early specialisation which is practised in our schools. It is wrong that a boy of 16 or 17 should spend all his time on mathematics, physics, and chemistry as a prelude to taking up a course at an engineering college. He should be devoting at least half his time to history, economics and the writing and speaking of English. And, at the university, is there room in the syllabus for historical studies ? Many college authorities say No : the three-years' course is already sufficiently crowded. But could not some of the purely technical material be surrendered with advantage? Every architectural student, incidentally, finds time to study the history of architecture.

A most interesting experiment is going on at Newcastle. There, in the mechanical department of King's College, University of Durham, Professor Burstall has introduced engineering history as one of the third-year's subjects in a three-years' course. Students devote about one-sixth of their time during the final year to this study. They are encouraged to read widely and discriminatingly in the general history of engineering ; but more particularly they study intensively a particular historical theme and submit a thesis upon it. Topics studied during recent years have included the history of the steam locomotive, the steam turbine, windmills, centrifugal pumps—and even mowing machines and water closets. Old records and drawings are examined, relics are studied in the museums, and, if possible, a prototype machine is taken to pieces and reassembled. All through his work the student is expected to bear in mind the social and economic background of the age he is studying, and to trace the various evolutionary threads that have led to the development in question. It cannot be doubted that Professor Burstall's students will leave Newcastle with a far more mature and balanced outlook than their contemporary graduates who have followed a more orthodox course.

But an engineer's education is not confined to his college days. It continues throughout his life, and, even if his training has not included the study of history or the other humanities, there is no reason why he should not make good such deficiencies after he has qualified. There are university extension courses, museums, and libraries, all waiting to be called upon.

Many engineers belong to the Newcomen Society for the study of the history of engineering and technology. Some of its members have done great work in tracing the evolution and history of various aspects of engineering. Regular meetings are held at the Science Museum in London. But membership is small and the only provincial branch so far formed is in Birmingham. We need more of the Newcomen outlook in the profession to-day.

The Value of History to Engineers.—Every engineer, then, should cultivate a broad knowledge of general history, and of the history of science and engineering. He will appreciate, as the layman cannot, the tremendous influence which technology has had in determining the political and social life of the world, particularly over the past two hundred years. As an engineer, he will be the more successful if he is conscious of present knowledge as the outcome of past trends. Early difficulties should be understood and the reasons for successes or failures assessed. As a study in human behaviour, it is fascinating to analyse the mind and motives of such men as Leonardo, George Stephenson, or Faraday.

The history of the development of original ideas and the overcoming of obstacles is an inspiration to young engineers. The message of Samuel Smiles is by no means out of date, but we do need a modern Samuel Smiles who will interpret technological history in the modern idiom, with due regard to the current swing away

from the "great man" theory to the "social environment" theory of history.

But not only in his daily work will the engineer feel the value of historical knowledge. The historical outlook gives a sense of proportion; things that are important can be separated from the things of the moment. The historical mind is the detached mind. The historically-minded engineer will develop his own philosophy; it will amuse him to see how historical events have been interpreted according to the bias of various writers, and he will learn to sort out the facts from the propaganda. He will begin to grasp the social and political implications of technological progress, and will begin to feel a sense of responsibility for the future. He will become conscious of the part he is playing in the moulding of history.

THE CONSEQUENCE OF INVENTION.—If we accept the principle that engineers are responsible for every technological development and for its consequences, then the moral stature (if not the status) of the engineer immediately rises. For example, road accidents are directly the concern of the engineer; he is responsible for the design of the roads and for the vehicles themselves; morally he is responsible for the consequences of his inventions. It is good that the Engineers' Guild has concerned itself with road safety matters.

Equally, engineers of all countries are partly responsible for the new nuclear weapons; without the engineer a hydrogen bomb could not be made—neither for that matter could aircraft or submarines. But engineers have yielded up their responsibility for the use of these things to politicians and soldiers. It is wrong that engineers should make them possible and then repudiate liability for their use.

The historically-minded engineer will, then, develop a responsible and deep concern for the future. He will be anxious to control to man's best advantage the development of nuclear power, automation in production, new methods of transport. He will be personally concerned that we do not squander our wasting natural resources of fuel and metals in inefficient and unnecessary ways. He will be deeply conscious of the problems of world population and food supply. Increasing the productivity of the earth is largely an engineer's problem; in his hands are the problems of irrigation, soil conservation, and the prevention of atmospheric and water-borne pollution. He will be conscious of the ugly legacy left us by our predecessors of the industrial revolution, and will be careful that we do not offend our descendants in the same way. He will accept his share of responsibility for the dehumanising of the worker which has come with mass production.

It is not suggested, of course, that we, as individuals, would be a penny the better off for possessing a knowledge of history. No employer is prepared to give an engineer a rise in salary because he can produce a diploma in history. But what individuals cannot accomplish for themselves, a whole profession can. We should look forward to a new generation of engineers, fully as competent professionally as those of today, but, in addition, deeply aware of the part played by science and technology in man's history. The Councils of the Institutions of Civil, Mechanical, and Electrical Engineers could then speak with a single voice for the profession. They could insist that, as this is an engineers' world, engineers should have a leading part in the directing of it. The technologist would no longer have to fight for the recognition of his status. It would be accorded to him as a right.

LECTURES ON BUILDING.

THE following lectures have been arranged by the Ministry of Works. Admission is free.

Weathering and Deterioration of Concrete and Cement Renderings, by C. Hobbs. College of Further Education, Park Square, Luton. November 20. 7.30 p.m.

The R.I.B.A. Form of Contract, by David Gardam. Gas Showrooms, Town Hall Extension, Manchester. November 19. 7.15 p.m.

Some Aids to Productivity, by A. E. Chittenden. Technical College, Roe Green, Hatfield. November 14. 7.30 p.m.

Application of Soil Mechanics to Buildings, by A. L. Little. London Polytechnic, Portland Hall, Little Titchfield Street, London, W.1. November 14. 7 p.m.

Safety in the Building Industry, by J. A. Hayward. Technical College, Hunny Hill, Newport, Isle of Wight. November 18. 7 p.m. And Education Offices, Guild Street, Burton-on-Trent. November 20. 7.15 p.m.

Safety in Excavations, by G. A. Roberts. Y.M.C.A., Blackett Street, Newcastle-upon-Tyne. November 19. 7 p.m.

New Traditional Methods of Building, by R. C. Scholefield. Brabloch Hotel, Renfrew Road, Paisley. November 19. 7.15 p.m.

Planning of High Flats, by T. L. Knight. College of Building, Cauldon Place, Stoke-on-Trent. November 19. 7.15 p.m.

Programming and Progressing for Builders, by A. E. Chittenden. New Marine & Technical College, South Shields. November 20. 7 p.m.

Sound Insulation, by J. A. Godfrey. Technical College, Bolton. November 21. 7.15 p.m.

Control of Concrete Quality on Sites, by R. A. Kenny. School of Building, Selhurst Road, London, S.E.25. November 21. 7 p.m.

Powered Hand Tools, by A. F. Coare. Technical College, Shorncliffe Road, Folkestone. November 25. 7 p.m.

Building in South America, by C. Graham. Park Hotel, Park Street, Cardiff. November 26. 7 p.m.

Common Defects in Buildings, by H. M. Llewellyn. Lauder's Restaurant, King Street, Kilmarnock. November 26. 7.15 p.m. And by H. J. Eldridge. Harris College, Corporation Street, Preston. November 27. 7 p.m.

Problems of Plastering and Rendering, by E. L. Westbrook. Technical College, Newstead Road, Weymouth. November 26. 7.15 p.m.

Essentials of Good Concreting, by E. E. H. Bate. Department of Building, 159 Tadcaster Road, York. November 27. 7 p.m.

Surface Finishes of Concrete, by J. G. Wilson. Technical College, Cherry Street, Stafford. November 28. 7.15 p.m.

Arbitration in Building Disputes, by Norman P. Greig. Technical College, Hempstead Road, Watford. November 28. 7 p.m.

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